OPTIMAL DAMPING FOR A SUSPENDED STRUCTURE UNDER LATERAL LOADING

by

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Abstract

The design of structural damping systems is an essential component in the design effort for a civil structure. This thesis examines the design and optimization of damping schemes to dissipate the energy input on a suspended structure due to lateral loading. The specific building examined is the suspended structure that was designed by Structures Group A for the purpose of the Master of Engineering High Performance Structures Group Project, Spring, 1998. Specifically, the effect on the building of earthquake loading will be examined. Due to the height of the building, earthquake loads controlled the lateral design. The methodology contained in this report can easily be applied to wind loading also.

The design of the damping schemes includes member sizes for the internal lateral bracing for the suspended structure and sizes for the required viscous dampers. The building is analyzed both statically and dynamically to withstand lateral loading in excess of what is associated with Eastern Massachusetts. 3-dimensional computer models that were utilized to approximate the behavior of the structure are included. A rough cost estimate is provided to assist with the comparison of alternatives and to furnish a general idea as to the expense of adding artificial damping to a structure.

Thesis Supervisor: Professor Jerome J. Connor
Department of Civil and Environmental Engineering

ACKNOWLEDGMENTS

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I never would have made it through this year at MIT without the support and encouragement of my family. So last but not least, I would like to thank my parents, Joseph and Susan Kmetz, and my girlfriend, Christine Dowling, for always being there.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1 Introduction

1.1 Purpose

This thesis examines the design and optimization of damping schemes to dissipate the energy input on a suspended structure due to lateral loading. The specific building examined is the suspended structure that was designed by Structures Group A for the purpose of the Master of Engineering High Performance Structures Group Project, Spring, 1998. Specifically, the effect on the building of earthquake loading will be examined. Due to the height of the building, earthquake loads controlled the lateral design. The methodology contained in this report can easily be applied to wind loading also.

1.2 Scope

The design of the damping schemes includes member sizes for the internal lateral bracing for the suspended structure and sizes for the required viscous dampers. The building is analyzed both statically and dynamically to withstand lateral loading in excess of what is associated with Eastern Massachusetts. 3-dimensional computer models that were utilized to approximate the behavior of the structure are included. A rough cost estimate is provided to assist with the comparison of alternatives and to furnish a general idea as to the expense of adding artificial damping to a structure.

2 Background

2.1 Structural Damping

The design of structural damping systems is an essential component in the design effort for a civil structure. It can not be dismissed as less important than the design of gravity load or architectural systems. The idea of introducing a separate system to artificially increase the damping in buildings has gained widespread acceptance over the past twenty-five years. Damping systems have been introduced into tall buildings, such as the World Trade Center and Citicorp Center in New York City, to combat lateral motion due to wind loading. They have also been utilized throughout the world to dissipate the destructive energy from earthquakes [1].

Dampers have been widely accepted and utilized to dissipate energy by many other industries in the past. The military is an example of one such industry. The recoil energy, from cannons that are bolted down to the decks of ships, has been dissipated by the implementation of dampers. When a round is fired from a gun, the forward momentum of the bullet is countered by the backward momentum, or recoil, of the gun. Without damping, the recoil force of a cannon would either rip the deck of a ship apart or severely bend it out of shape. A damper placed between the gun and the ship, however, dissipates the majority of the recoil energy and prevents damage [2].

Over the last five years, the idea of using seismic dampers has become more increasingly accepted. Seismic dampers are devices that are independent of a structure's gravity frame and can easily be installed in both new buildings and retrofits of old buildings [2]. Since seismic dampers have only been considered viable solutions in civil engineering for the last five years, they can be classified as an "emerging technology". For this reason, the addition of viscous dampers to a building would not only alleviate the problem of motion due to lateral loading, but would also help further classify the building as a "High

Performance Structure". In this context, the phrase "High Performance Structure" is used to describe a building that utilizes the latest innovations to meet the desired performance requirements. These requirements could range from motion restrictions to intelligent control systems for monitoring the internal environment of the structure.

Introducing a structural damping system into a building essentially increases the effective damping of the structure artificially. Before a building can be analyzed, a level of inherent damping must be assumed. The actual damping present in buildings is difficult to measure and varies depending on the type of structural system, cladding system, and materials used for construction. The inherent damping in a structure can not be stated with the same degree of accuracy as other dynamic characteristics [3]. The mass and natural periods of a structure are two such examples.

The amount of actual damping present in a structure has a significant effect on its dynamic response. Changes of assumed damping from 1% effective to 2% effective can reduce the root-mean-squared acceleration response of a building by as much as 20 to 30% near resonant conditions [3]. This fact results in a problem when analyzing a structure dynamically. Since a structure's intrinsic damping can not be found accurately, the perceived response of the structure due to lateral loading will have a large uncertainty associated with it. If the level of response can not be accurately gauged, an appropriate solution technique will be difficult to find. The uncertainty associated with estimating the natural damping can be alleviated by introducing energy dissipating systems into the design of buildings to provide specified amounts of damping [3].

2.2 Conventional Theory

In the 1950's, George Housner presented papers in which he showed how a building absorbs the effects of an earthquake. The phenomenon, known as hysteretic damping, requires that certain parts of the frame, such as the beam-to-column joints, yield [2]. For the past few decades, the seismic response of structures has been thought of this way. There are clearly benefits to having hysteretic behavior in a structure. If certain well-positioned members were designed to absorb the majority of the seismic energy by deforming, then only those components of the structure would need to be repaired or replaced after an earthquake.

The problem is how to actually build a structure that exhibits reliable hysteretic behavior. If the steel frame connections are used as the hysteretic elements, then the building's gravity load bearing system would also be required to resist the applied lateral loads. Since the members are actually supposed to absorb the earthquake energy by yielding plastically, there will be residual deformation in the connections. The gravity load system also has the potential to fail under extreme seismic loading. A structure that only has one system to resist both gravity and lateral loads lacks redundancy. When the system fails, significant damage to the structure, and possibly irreparable failure, results.

In the past, it was believed that a building would perform better by adding more steel to it. This, in effect, increases the strength and stiffness of the structure. The problem is that the mass is also increased and the period is consequently reduced. The force a structure experiences under seismic loading is a result of the random motion of the ground that supports the structure. Structures with lower natural periods respond more to ground excitation because the forcing frequencies are closer to the building's natural frequencies. The ground motion produces inertia forces, which vary with the mass of the structure and the magnitude of the ground acceleration. Therefore, the heavier of two structures will experience a greater inertia force due to the same earthquake. This additional force must then be dissipated.

The 1971 San Fernando earthquake showed that steel buildings designed using modern building codes would probably perform well in earthquakes. The revisions to the code after San Fernando reflected the thinking that stiffer, stronger buildings were better [2]. However, the 1994 Northridge earthquake has exposed flaws in this reasoning. The newer buildings, which incorporated steel moment frames into their designs, suffered considerable damage. The beam-to-column joints that were supposed to yield failed in a brittle manner instead, and did not allow for the presumed hysteretic energy dissipation capabilities [2]. Obviously, the conventional theory on earthquake connection design needs some modification.

3 Suspended Structure

3.1 Structures Group A Design Project

This thesis examines the design and optimization of damping schemes to dissipate the energy input on a suspended structure due to lateral loading. The specific building to be examined is the suspended structure that was designed by Structures Group A for the purpose of the Master of Engineering High Performance Structures Group Project [4]. See Figure 3-1 for a rendering of the building created in AutoCAD. A brief description of the building is outlined below. The structural layout, member section types, and members sizes are included, as this information was essential in completing a dynamic analysis of the suspended structure.

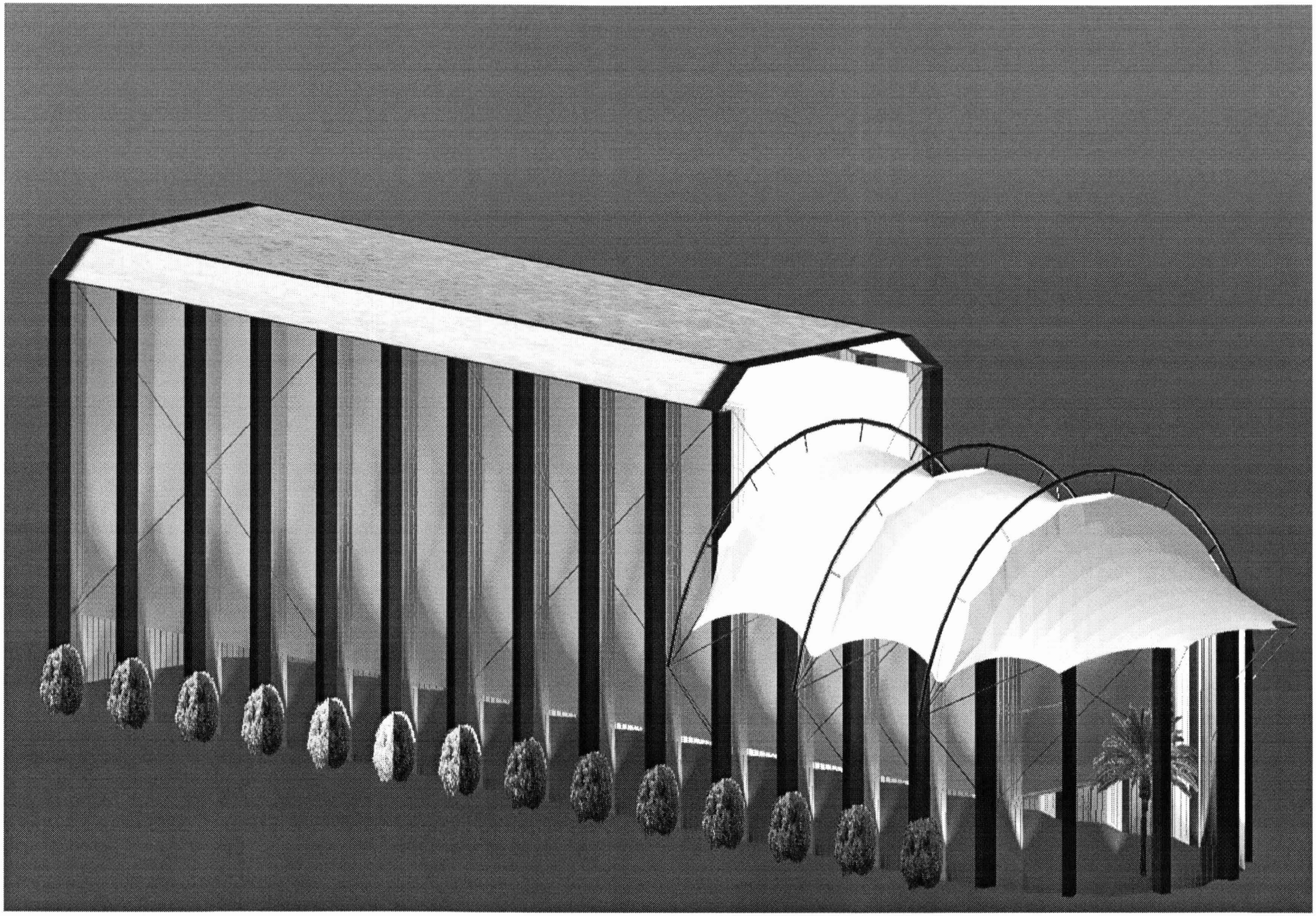

Figure 3-1: Southeast isometric rendering

3.2 *Description*

For the building to meet the minimum prerequisite of 110 thousand square feet of space, the structure required a footprint of 80 feet by 400 feet. This space is spread out over the basement, ground level floor, and the four suspended floors. The first four bays, or 100 feet, on the East End of the building will only support two floors and will rise 70 feet. The remaining bays have exterior columns, spaced 25-foot on center, that rise 100 feet unbraced to a deep truss. The truss spans 80 feet across to another column on the opposite side of the building. See Figure 3-2 for an architectural drawing of the building.

The main building structure is 60 feet wide and is supported only at the edges by the truss above. Each floor is supported by a cable connected in series to the floor above it and eventually to the truss. The ground floor and the basement are an independent structure. The exterior columns support an outer façade of glass that, in addition to creating a tall atrium, functions as the building envelope. The wind loads travel directly from the glass façade down the exterior columns, thus bypassing the suspended structure.

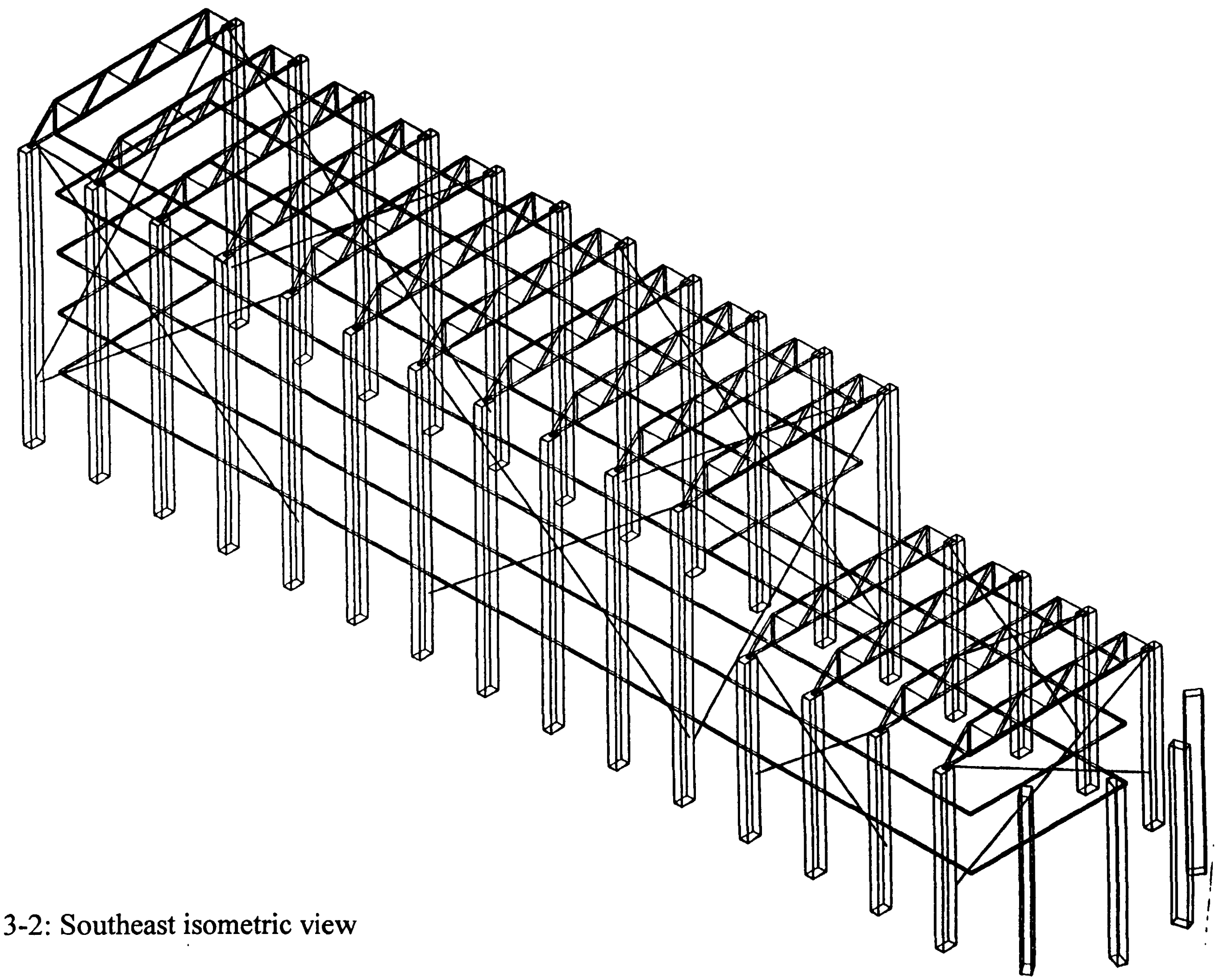

Figure 3-2: Southeast isometric view

3.3 Structural Layout

The building is supported by structural frames spaced every 25 feet on center along the long axis of the building. Each structural bay consists of two columns with a 100-foot unbraced length supporting an 80-foot long deep truss. The truss is 10 feet deep and will be covered by a corrugated steel deck roof. The bottoms of the trusses are flush with a concrete slab that will support the majority of the mechanical equipment for the building.

Ten feet inside the building, two high-strength steel cables are affixed to the truss and connected to the top floor of the building. The cables connect to deep steel girders that support the individual floors. Each girder spans 60 feet, across the building, to the connection with a cable on the opposite side. The girders support steel beams spaced every 10 feet on center. These beams span 25 feet between girders and support the floor. The floor consists of a rolled steel deck with a concrete slab. See Figure 3-3 for a detailed Structural Drawing of a typical frame of the building.

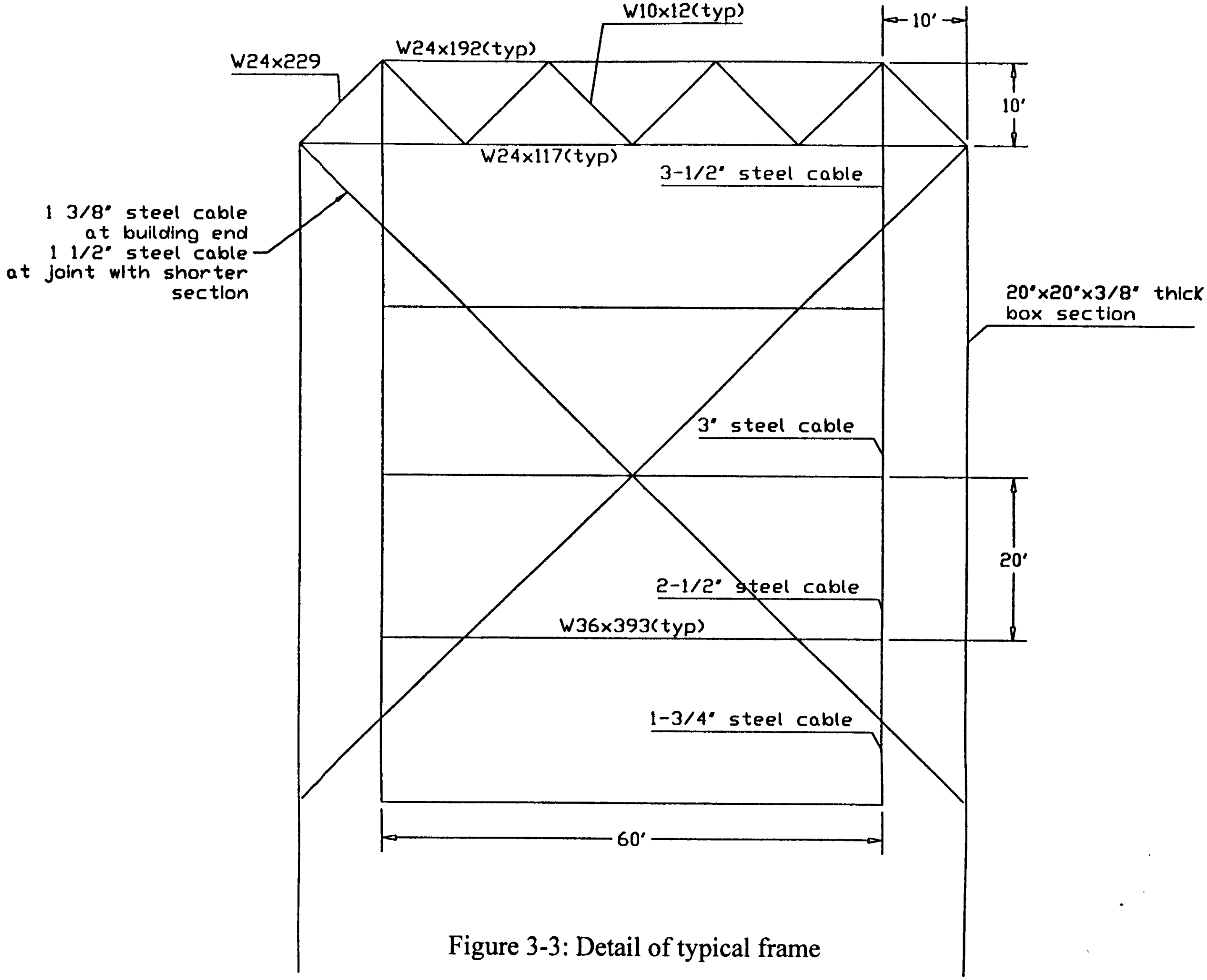

Figure 3-3: Detail of typical frame

3.4 Member Sizes

The members were designed in accordance with the Load and Resistance Factor Design Manual 2nd Edition [5]. An educational version of SAP2000 was utilized for the analysis and design of the wide flange and steel cable sections. All steel sections are 50ksi. The cables have a design strength of 120~130ksi.

Slab

The slab is made from lightweight concrete and is 5 inches thick. It is poured over a 3 inch, 16-gauge, corrugated steel deck. This specification allows for the required live load of 200-lbs/ft^2. The dead weight of the slab and deck is 34 lb/ft^2.

Beams

The beams support the distributed loads of the slab and carry it to the girders. The spacing between the beams is 10 feet and they span 25 feet to the girders. Therefore, the distributed dead load is 0.34 kips/ft from the slab plus its self-weight and the live load is 2 kips/ft. The required beam size is a W18x50 rolled steel section. To minimize the negative moment effect on the slab, the beam-girder connections are pinned.

Girders

The girders support the concentrated load from the beams every 10 feet and carry it to the steel cables on the outside of the suspended structure. The connections to the cables are also pinned because cables are unable to carry moment. The required girder size is a W36x393 rolled steel section.

Interior Cables

The girders are supported on the ends by high-strength steel cables. The fifth floor cables required the largest cross section since they are supporting the most weight. Each floor adds an additional factored load of 255 kips. The diameters of the interior cables range from 1 ¾ inches for the second floor to 3 ½ inches for the fifth floor.

Truss

The truss was designed using W sections. The largest member sizes were limited to 24 inches in depth. A drawing of the truss with the member sizes defined can be found within Figure 3-3.

Exterior Columns

The exterior columns were designed to support an axial load that included the weight of the truss, the weight of the suspended frame, and the reduced live load. Box sections were chosen over wide flange sections for aesthetic purposes. The required column dimensions, for the five-story portion of the building, were 20" x 20" – $^{3}/_{8}$" thick. The columns required for the three-story section were 20" x 8" – $^{5}/_{16}$" thick.

3.5 Lateral Load Requirements

After the design of the members was complete, the response of the structure due to lateral loading was examined. The suspended structure required both internal floor-to-floor bracing and dampers. The steel bracing was required because the girders are attached to the truss by cables, and the cables do not provide the structure with any lateral stiffness.

Due to the height of the building, seismic loading controlled the design of the lateral load system. The natural period of the suspended structure and external columns was initially estimated to be in the region of 1 second. This value corresponds to the frequency excitation range for earthquakes. Wind loads tend to have higher periods, in the range of 5 to 6 seconds. For this reason, wind loads would not be apt to excite this building and cause large displacements. Seismic loads, on the other hand, will cause the building to hit resonance. Dampers must be installed to reduce the motion of this structure.

4 Internal Lateral Bracing

4.1 *Justification*

The suspended structure required additional floor-to-floor stiffness in the form of steel bracing members. This was because the members attaching the floors to the roof trusses were steel cables and therefore they do not provide the required level of stiffness. The amount of steel required per floor was calculated by first using the ASCE Minimum Design Loads for Buildings and Other Structures to calculate the effective, statically applied, earthquake loads at each level of the building [6]. Sufficient bracing was then installed to absorb these forces.

The braces are located along the outside corners of the building. They will be concealed by enclosing them in the walls surrounding the elevator shafts. Double angle sections were selected for the steel members because the amount of steel required was not large enough to warrant using rolled sections. High-strength steel cables could also have been used, but since the cables must be pre-stressed, they would have increased the stress in the structure. The fact that double angle sections do work in compression, unlike cables, also influenced the decision.

4.2 *Bracing Design*

The maximum shear between floors, for an entire floor, was calculated to be 163.5 kips at the base of the structure. Due to the unknown effects of the dampers, to be installed at the base of the suspended structure, at this point in the design, all floors are designed to carry this maximum shear. Assuming the braces act only in tension, four braces is required per floor. The shear load is carried by two L2x2x$^3/_8$ members. Due to the symmetry of the building and loading possibilities, the same bracing is adequate for both directions. For constructability considerations, the same bracing is used in the short portion of the building.

The equations and constants required to calculate the lateral seismic force induced at the base of the structure and the calculations for the lateral bracing cable design can be found in Appendix A.

5 Damping

5.1 *Introduction*

Damping is the mechanism by which a system's energy is gradually converted into friction, heat, sound or other forms of mechanical energy. Due to the reduction in energy, the response of the system, such as the displacement, gradually decreases. Damping is especially effective at reducing the system response near resonance where it governs the response [7]. A damper is assumed to have neither mass nor elasticity, and damping force exists only if there is relative velocity between the two ends of the damper. Dampers can be divided into two categories: active and passive. Passive dampers can further be divided into the following categories: viscous, coulomb or dry friction, hysteretic, and visco-elastic. For the purpose of this thesis, the advantages and disadvantages of the various types of damping were compared and an alternative was chosen. Fluid viscous dampers, such as the ones developed by Taylor Devices, were chosen based on the following information.

5.2 *Active Damping Systems*

Active damping devices are classified as those that require outside power sources to perform. In addition to the energy source, a method of monitoring the response is necessary. Active control entails monitoring the input and/or output, and adjusting the appropriate parameters, to bring the actual response closer to the desired response. The parameters available to be altered include the input and, in certain instances, the system itself [7].

The two methods currently utilized to actively adjust systems are open-loop control and feedback, or closed-loop, control. Open-loop control refers to the technique of monitoring the input to a system and then adjusting the loading appropriately. Feedback control is the method where the response is observed and the resulting information is used to apply corrections to the loading of the system.

In addition to applying a correction to the input, the control system may also adjust certain properties of the actual system. An example of this would be changing the stiffness of a structure by repositioning the structural components in real time. The ideal active control system would be able to continuously assess its performance with respect to a given criterion, and adjust its properties accordingly so as to maintain optimal performance at all times [7]. This scenario, which is referred to as adaptive control, has not yet been realized in an economical manner.

The basic components of an active control system are the sensors, controller, and actuators. The sensors are the measuring devices. For a structure, they can gauge any number of variables including displacement, velocity, acceleration, and even external forcing parameters such as pressure and ground motion [7]. The measurement of ground motion would be especially pertinent to utilizing active control to counter the effects of seismic loading on civil structures. The function of the controller is to compare the observed response to the desired response, ascertain the best control action, and transmit the appropriate commands to the actuators [7]. The actuators then apply the necessary action, such as a force, to the system.

Active control systems perform better when compared with passive control devices because they are able to quickly conform to changes in the surroundings, but they also have disadvantages when considered for civil structures. The design and implementation of large-scale control systems are complex. It is difficult to maintain infrequently used mechanical systems. The control devices must be capable of generating large control forces with high velocities and fast reaction times. The fact that power is not always available during an earthquake requires having a convenient auxiliary power supply to activate the system. In general, active control systems are also more expensive than passive systems [7].

5.3 *Passive Damping Systems*

Passive damping systems require no outside power source to operate. Unlike active control systems, once passive systems are installed they can not be modified instantaneously to compensate for an unexpected loading [7]. This fact tends to result in an over-conservative design. Since passive control systems are generally less expensive than active control systems and an external energy supply is not required, they are currently more prevalent in structural systems. The four subcategories of passive devices are discussed below.

Fluid Viscous Damping

Fluid viscous dampers operate on the principle of fluid flow through orifices. They consist of pistons in metal cylinders filled with silicon fluid and work like shock absorbers [2]. A typical fluid viscous damper is displayed in Figure 5-1. The viscous damper's output force is resistive. Therefore, it acts in a direction opposite to that of the input motion. The amount of dissipated energy depends on many factors, such as the viscosity of the fluid, the frequency of vibration, and the velocity of the vibrating body.

The advantages of using fluid viscous dampers over the other various options are that fluid dampers are self-contained and do not require auxiliary equipment or power, small, compact, and easy to install, and inexpensive to purchase, install, and maintain. Fluid viscous damping also reduces the internal shear forces in the structure, as well as the deflections, and allows the structure to restore itself to its original position after seismic events [8].

Visco-elastic Damping

Visco-elastic devices are stacked plates separated by inert polymer materials [2]. A typical visco-elastic damper is displayed in Figure 5-2. Visco-elastic devices have an output that is somewhere between that of a damper and a spring [9]. The damper's properties vary with temperature and the excitation frequency. One of the most serious problems with these devices is the fact that their performance varies significantly with temperature. As the temperature increases, the viscosity of the fluid in the dampers decreases. If the fluid is too thin, the damper will not provide sufficient damping under loading. This fact prevents visco-elastic dampers from being the optimal choice for external use in regions that experience large temperature fluctuations throughout the year.

Another problem is that under high level seismic inputs, the spring response dominates. This produces a stiffer system that responds more to the seismic input. In the past, the stiffness of buildings was increased, by adding more steel, to combat the effects of an earthquake. This solution technique proved to be inadequate during the 1994 Northridge earthquake. The newer, stiffer, stronger buildings, which utilized steel moment frames, suffered considerable damage [2]. This proves that a stiffer building does not absorb the earthquake loads as was first believed, but actually responds more to an earthquake. For this reason, the stiffness associated with visco-elastic dampers is not a favorable trait.

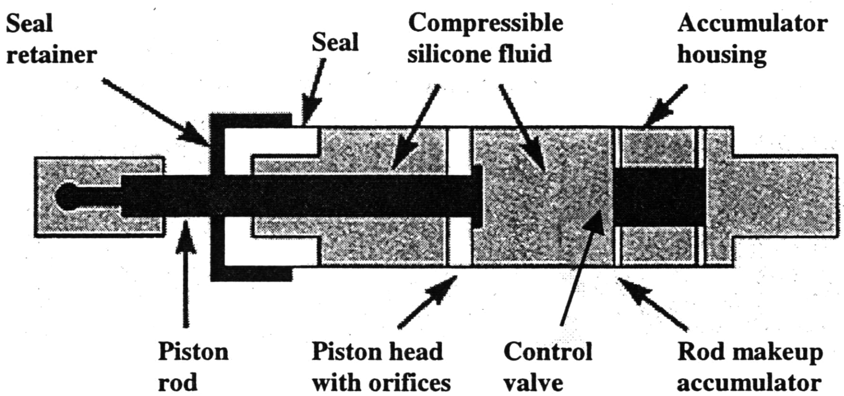

Figure 5-1: Typical viscous damper

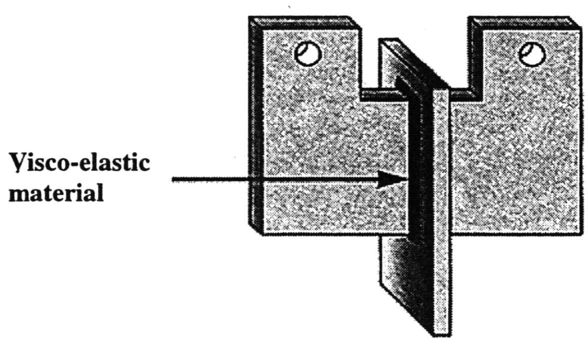

Figure 5-2: Typical visco-elastic damper

Hysteretic Damping

Hysteretic damping deals with the case where certain members actually absorb and dissipate energy by deforming. A typical hysteretic damper, with curved steel plates as the yielding members, is displayed in Figure 5-3. The effect is due to friction between the internal planes, which slip or slide as the deformations take place. When a body that exhibits hysteretic damping is subjected to vibration, the stress-strain diagram shows a hysteresis loop. The area of this loop denotes the energy lost per unit volume of the body per cycle due to damping [10]. The disadvantage of hysteretic dampers is that the deforming members have the possibility of failing under repeated loading. If the members that yield also comprise the gravity load bearing system of the building, as was the case in the past, then failure of these members would be detrimental to the survival of the structure.

Coulomb Damping

A typical coulomb or dry friction device, with sliding steel plates absorbing the energy, is displayed in Figure 5-4. Coulomb or dry friction dampers provide a constant force in magnitude. The direction of the force is opposite to the direction of the vibrating body. The energy dissipation is associated with overcoming the friction between rubbing surfaces that are either dry or have insufficient lubrication. The disadvantages of dry friction dampers are that the magnitude of the force is constant and that these devices restrict a structure from restoring itself to its original position after seismic events.

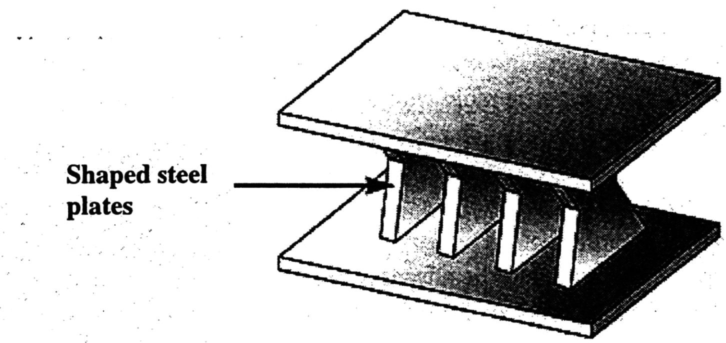

Figure 5-3: Typical hysteretic damper

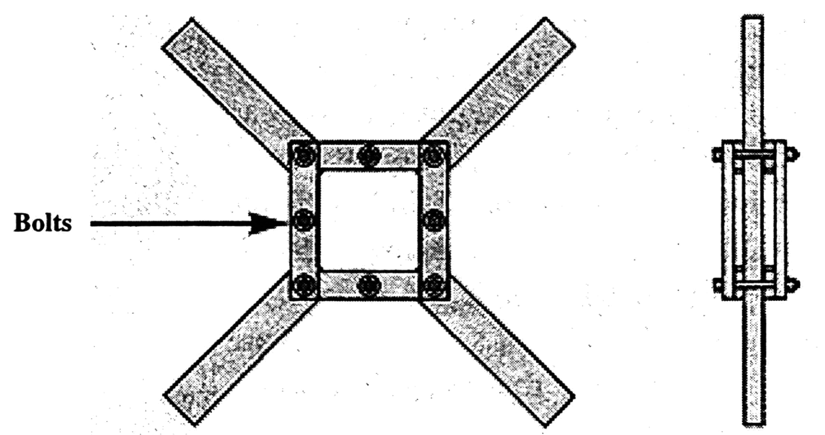

Figure 5-4: Typical friction damper

5.4 *Recommendation*

Fluid viscous dampers were selected as the optimal choice for the damping elements to be installed in the suspended structure to combat lateral motion. The fact that these dampers are excellent at dissipating potentially destructive energy, while remaining relatively inexpensive to purchase, was a major factor. Construction issues are addressed by the ease of installation and low level of required maintenance. Fluid dampers are also virtually unaffected by temperature variations within the range of -40° to +160° Fahrenheit [9].

Fluid viscous dampers also represent an "emerging technology" in civil engineering. This coincides with the desire to produce a design for a "high performance structure" that utilizes the latest innovations to meet the desired performance requirements. The decision to utilize fluid viscous dampers in the design was based upon the combination of all of the aforementioned reasons.

6 SAP2000 Analysis

6.1 Model

The first step was to create a three-dimensional model of a typical frame of the building in SAP2000. Due to the complexity of the suspended structure, it was not reasonable to assume that the building would behave in a manner similar to that of a shear beam. An accurate computer model was the only feasible way to determine what the response of the suspended structure would be to various loading conditions. SAP2000 was actually utilized to design the sizes of the necessary members as well as ascertain a reasonable approximation of the behavior of the structure.

An illustration of the model is shown in Figure 6-1. A typical frame of the 5-story section of the building was modeled at first. The floor slabs are assumed rigid and the effective contribution of all bracing was calculated. The structure was split into two buildings, due to the height difference, for the purpose of this analysis. An expansion joint will be placed between the two sections of the building to create the desired scenario.

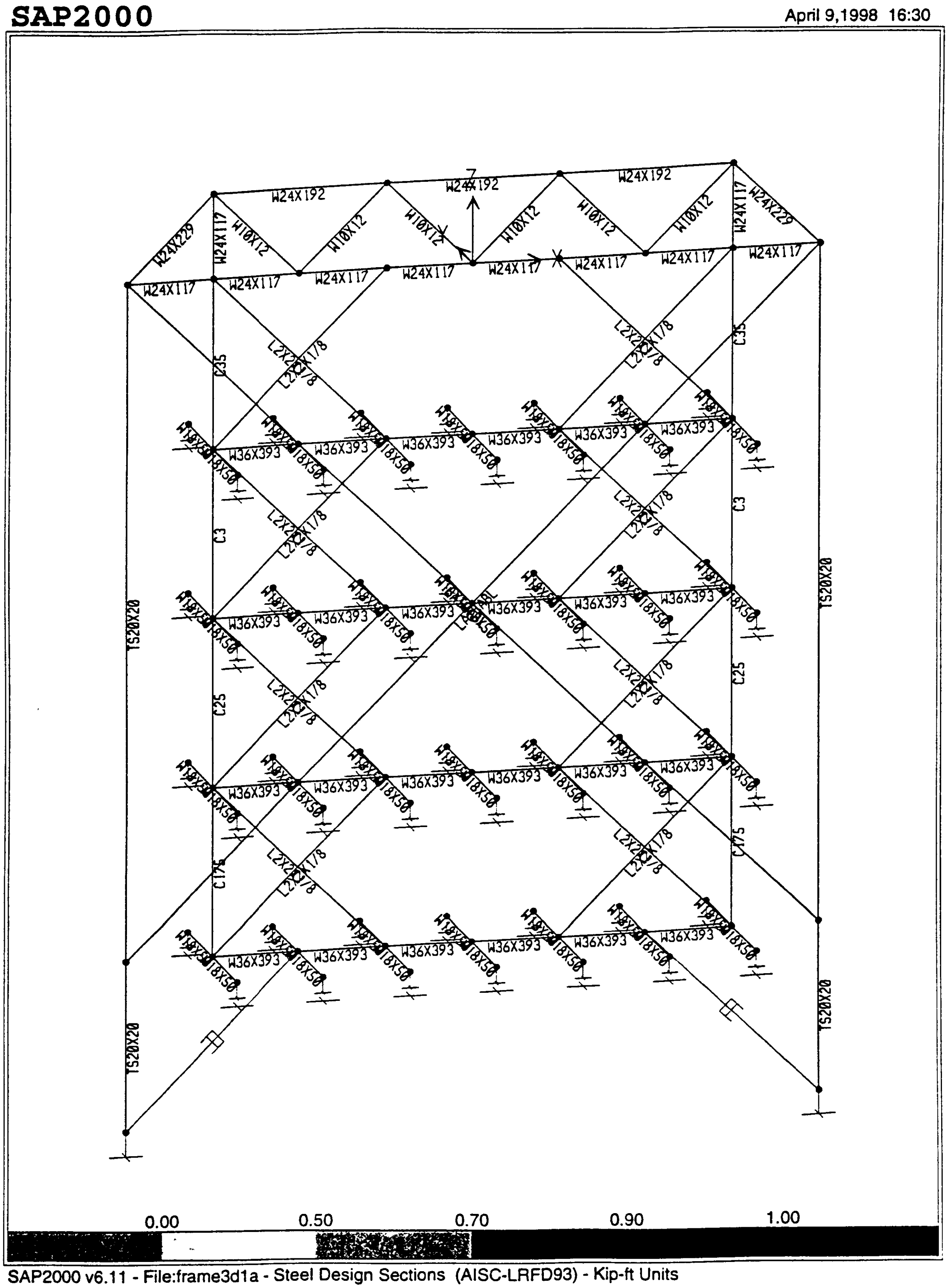

Figure 6-1: SAP2000 model of structural frame

A three-dimensional model of the structure was necessary since SAP2000 was used for the design of the members also. For the purpose of the analyses, though, the building was only allowed to displace in the two-dimensional x-z plane. The reason for this was to eliminate the out of plane modes that would only serve to complicate the analysis. The earthquakes were applied in the x-direction and the corresponding lateral motion was the desired output.

The geometry of a typical frame of the structure was entered into SAP2000 and the program was then utilized to assist in selecting the appropriate member sizes. The tributary area of the frame was incorporated by including half of the beams on either side of the girders. The dead load of the slab and the 200-psf live load were both applied to the beams. The moments were released at the ends of the beam members. The exterior columns were classified as having a fixed boundary condition at the base. High strength steel was used for the interior cables. The frame was first designed to withstand gravity loads.

6.2 Seismic Loading

An earthquake can be classified as a random event. The magnitude and frequency content can not be accurately predicted and two earthquakes will never be identical. For this reason, designing a structure to withstand one particular earthquake does not assure the structure can withstand all seismic loading scenarios. Two common earthquakes, with different frequency contents, were utilized for the analysis of the suspended structure. The lateral load system was designed to withstand the ground accelerations due to both El Centro and Northridge. It is reasonable to assume that if the system can adequately damp motions due to both earthquakes, it will be successful under other loading conditions within the same seismic zone.

The force a structure experiences under seismic loading is a result of the random motion of the ground that supports the structure. The ground motion produces inertia forces, which vary with the mass of the structure and the magnitude of the ground acceleration. Therefore, the heavier of two structures will experience a greater inertia force due to the same earthquake. Since the applied load is also dependent upon the magnitude of the ground acceleration, the acceleration data must be appropriately scaled depending upon the region of interest. The seismic regions, or zones, are divided geographically and are based upon previously collected earthquake data. The scaling factors are determined by the magnitude and occurrence rate of earthquakes in each zone. They can also be increased or decreased depending on the expected use of the structure. Obviously, a building to be utilized for the design and production of microelectronics would require more stringent criteria than other structures.

A time-history analysis was performed on the 5-story frame using SAP2000. A time-history is a record of the ground acceleration at defined time segments for a specific earthquake [11]. The assumption that the building responds similarly in the x- and y-directions was made due to symmetry. Two earthquakes, El Centro and Northridge, were therefore applied to the building in only the x-direction. Plots of 60 seconds of acceleration data for both earthquakes are shown in Figure 6-2 and Figure 6-3. The magnitudes of the earthquakes were initially scaled down according to the Massachusetts State Code for the seismic zone that encompasses Eastern Massachusetts [12]. After some consideration, it was decided that a higher level of ground acceleration would be applied. This was done to insure that the response of the structure would be significant. A larger response makes for a more interesting design problem and proves that fluid viscous dampers can be utilized in high seismic regions, such as California.

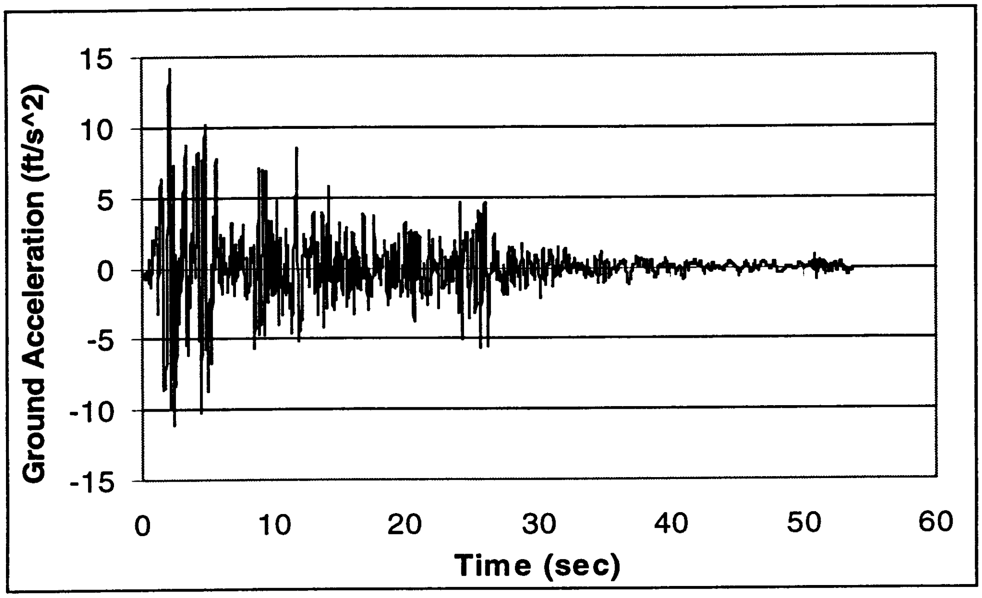

Figure 6-2: El Centro ground acceleration versus time

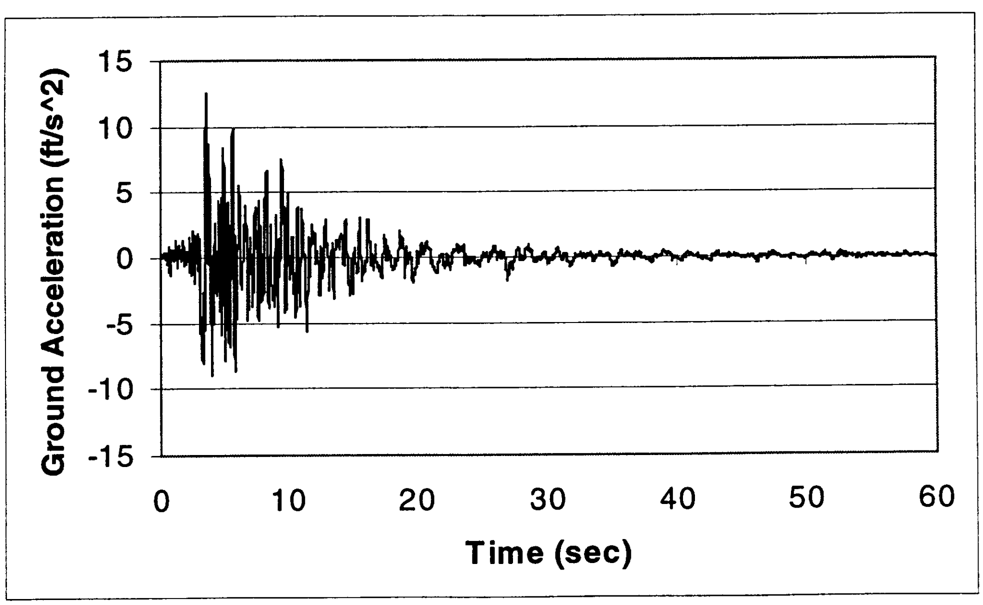

Figure 6-3: Northridge ground acceleration versus time

6.3 *Analysis Methodology*

Once the model was created in SAP2000, the first step was to check the shear force, bending moment, and axial force diagrams of the various components. See Appendix B for representations of the model with these items displayed. All of the diagrams agree with the expected results. The girders carry the largest values of shear and moment due to the applied dead and live loads from the slab and beams. The beams are pin connected to the girders, and therefore only carry shear. The interior cables and the lateral bracing only carry axial forces.

A linear transient analysis was performed on the building once the earthquakes were entered into SAP2000 in an acceptable format. Due to the analysis type chosen, the structure began with zero initial conditions and all elements were assumed to behave linearly for the duration of the analysis. The intrinsic damping present in the structure was estimated to be 0.5%. This value, while slightly low, is a reasonable assumption for most buildings. It also prevents the design from being overly dependent upon the inherent damping to absorb a significant amount of input energy.

Graphs were made of the input energy, base shear, and certain nodal displacements versus time for each earthquake. Node numbers 5 and 18 were chosen to examine the displacement of the structure. Node 5 is located at the top of the right exterior column. Node 18 is located on the bottom, right hand side of the suspended portion of the structure (see SAP drawings in Appendix B).

The natural mode shapes and periods of the suspended structure were also analyzed. See Figure 6-4 for a diagram of the first mode shape. See Appendix B for pictures of the other mode shapes. The lowest modes of a structure are the most easily excited. Therefore, the response of the suspended structure can be characterized by the displayed shapes. Notice that the first two mode shapes resemble the deflected shape of the building due to lateral loading. The fourth mode shape coincides with the deformed shape due to gravity loading. One can assume that these modes will be stimulated by the corresponding loading conditions.

The natural periods of this structure are in the range of 0.27 to 2.83 seconds. Seismic loads have periods of about 1 second, while wind loads are approximately 5 to 6 seconds. The natural periods, and hence frequencies, of the lowest modes of the building overlap the forcing frequencies of earthquakes. Therefore, it can be concluded that earthquake loading controls the lateral design of this structure over wind loading. This is not surprising because wind loading is usually more critical for tall structures.

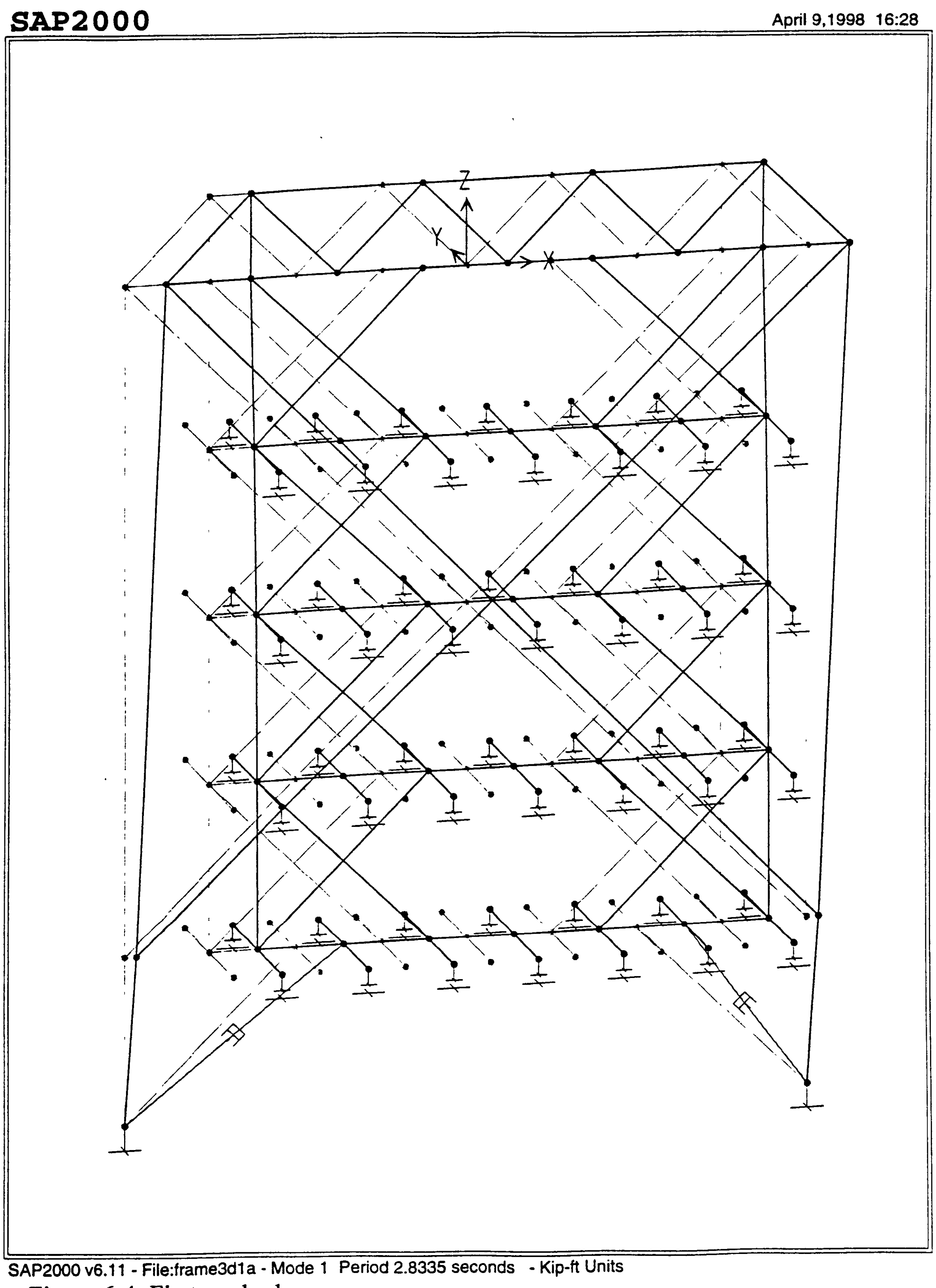

Figure 6-4: First mode shape

6.4 *Results*

By examining the plots of displacement versus time for the nodes mentioned above, a general idea of the response of the structure to each of the earthquakes can be ascertained. El Centro caused a maximum displacement of 24.4 inches at the base of the suspended structure (node 18) and 22.5 inches at the base of the truss (node 5). The imposed accelerations due to Northridge caused much smaller displacements throughout the structure. The maximum displacement was 10.6 inches at node 18 and 9.8 inches at node 5. The frequency contents of the two earthquakes account for the reason El Centro caused a larger response in the building than Northridge. El Centro's predominant forcing frequency was closer to the natural frequency of the lower modes of the suspended structure.

To reduce the motion of the structure, two dampers were inserted into the model. The dampers are attached, at an angle of 45°, from the ground to the bottom of the suspended portion of the building (See Appendix B). The reason this scheme was chosen is twofold. The largest lateral deflections were observed to occur at the base of the suspended structure, or the second floor of the building. In addition, the suspended structure behaves like a giant inverted pendulum that is pinned at the top, as opposed to a shear beam. Therefore, inter-story drift is not the main concern. For a building that can be modeled by a shear beam, the logical placement of the dampers would be within the floor-to-floor steel cross bracing. However, for a building where the floors are swaying, rather than sliding with respect to each other, there is no reason to place dampers within every floor.

As a "first cut", 30 kip dampers were used. In the actual design, this corresponds to two pairs of 150 kip dampers being installed in the two end frames. The motivation behind this approach is to avoid having to install dampers in every frame of the structure. This would compromise the aesthetic appeal of the suspended structure and possibly increase cost. By installing the dampers in the manner suggested, the problem of the motion of the structure will still be addressed without taking anything away from the design architecturally. The placement of the dampers should also bring more attention to the fact that the building is actually suspended from the roof trusses.

The results from the analysis of the model with the dampers included are displayed in Figure 6-5 and Figure 6-6. As can be seen from the graphs, the response of the structure to both earthquakes is dramatically reduced. The maximum displacement due to either El Centro or Northridge is reduced to less than 5 inches. These numbers can be compared to the previously found values of 2 feet for El Centro and 10 inches for Northridge. This represents an 80% reduction in the response due to El Centro and a 50% reduction in the response due to Northridge.

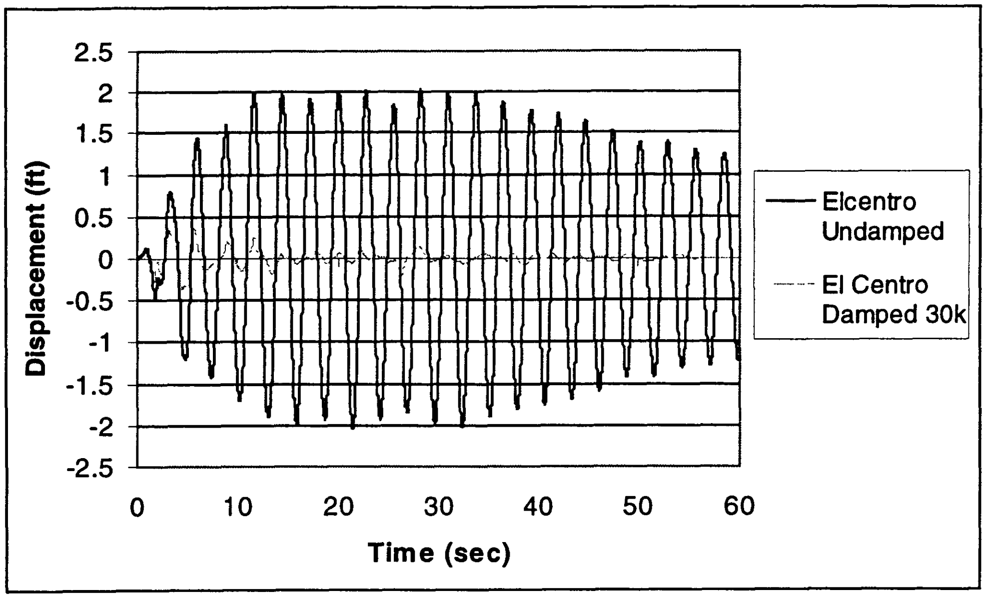

Figure 6-5: Displacement at bottom of the suspended structure due to El Centro

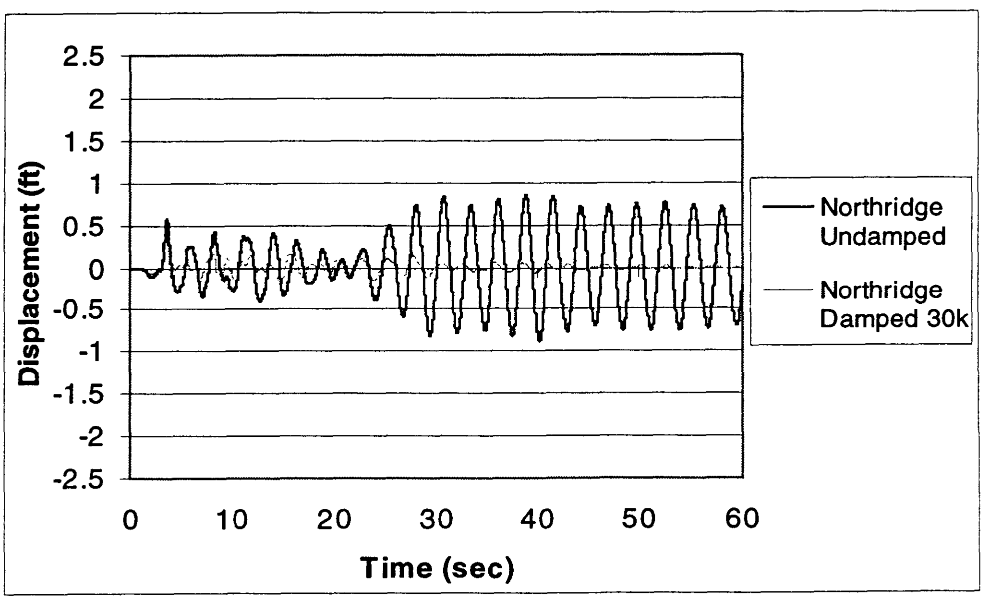

Figure 6-6: Displacement at bottom of the suspended structure due to Northridge

6.5 Additional Analysis

While these results are encouraging, especially at the top of the structure, there is still opportunity to further reduce the motion at the base of the suspended portion. For the second analysis, the viscous damping force was increased from 30 kips to 60 kips. This value corresponds to either the same configuration as before with 300 kip dampers or a new scheme where 150 kip dampers are installed in four frames. Eight dampers would then be required. The results of this analysis are shown in Figure 6-7 and Figure 6-8. The maximum displacement at joint 18 is further reduced to 2.8 inches for El Centro and 3.3 inches for Northridge. See Table 6-1 for a summary of the maximum displacements of the suspended structure for all three cases that were analyzed.

Earthquake	*Undamped (in)*	*Damped 30k (in)*	*Damped 60k (in)*
El Centro	24.4	4.45	2.76
Northridge	10.6	4.57	3.28

Table 6-1: Maximum displacements of suspended structure

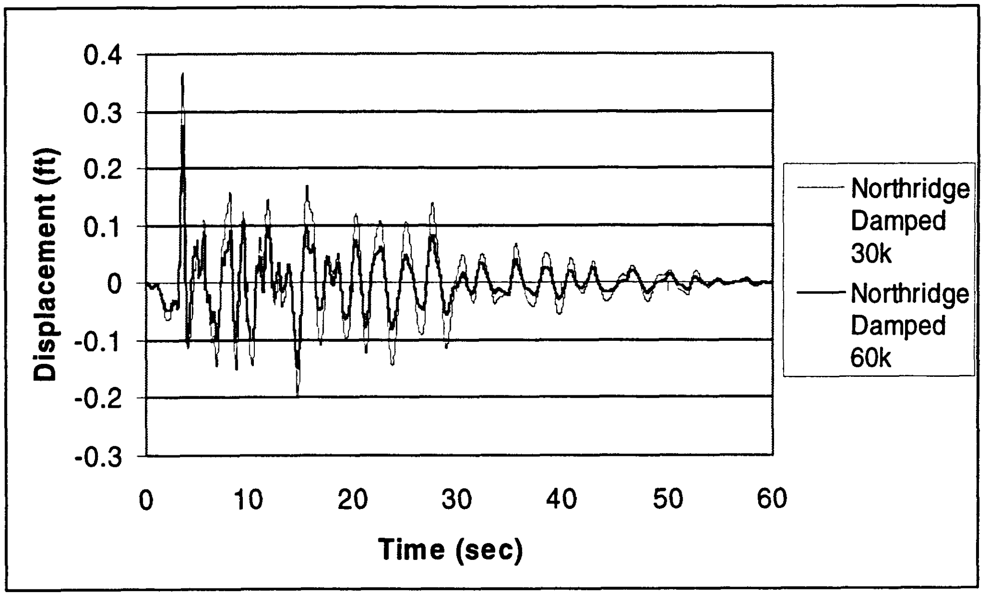

Figure 6-7: Damped displacement due to El Centro

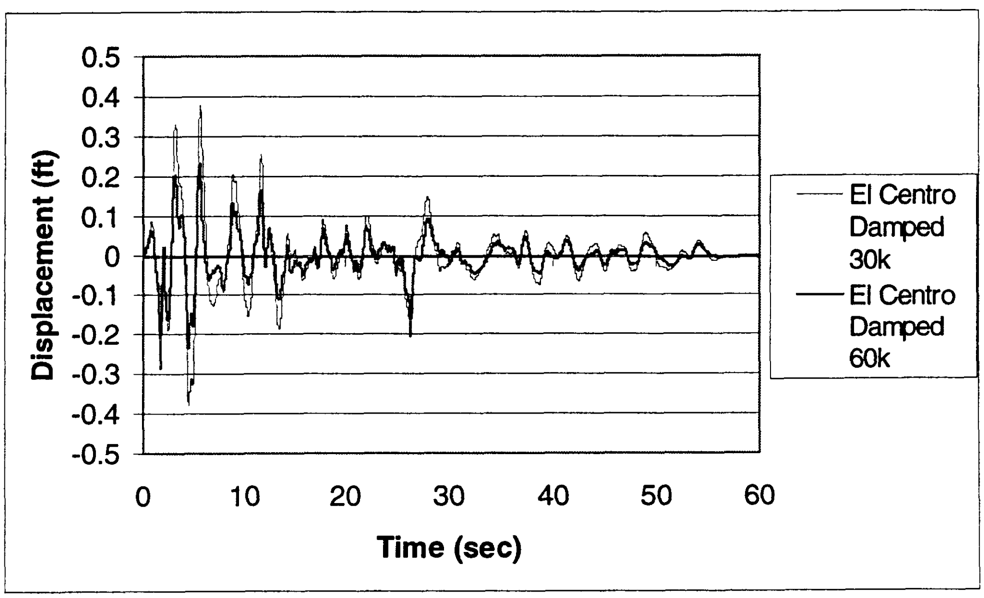

Figure 6-8: Damped displacement due to Northridge

A typical frame of the stepped-down portion of the building could not be analyzed, but less damping should be required in this section since it has less mass and stiffness than the 5-story section. The heavier and stiffer a structure, the more earthquake force it will attract and have to dissipate [2]. Therefore, if the amount of damping required in the 5-story section were applied to the 3-story section, the deflections would be reduced well below an acceptable level. For the purposes of calculating the cost of the damping system, it will be assumed that the same damping force, of 120 kips per frame, is required for the short section. Two 150 kip dampers will be installed in the two end frames of the 4-story section. The same size dampers could be used for constructability purposes. The actual response of this portion of the building should be examined in the future.

6.6 Base Shear

The addition of fluid viscous dampers to the suspended structure not only limited lateral deflections, but also yielded an additional benefit. The shear force at the base of the exterior box columns is also reduced significantly. Plots of the base shear versus time for the undamped and damped systems are shown in Figure 6-9 and Figure 6-10. The maximum base shear is lowered from 186 kips to 21.5 kips for El Centro and from 81 kips to 25 kips for Northridge. This is significant because before damping was introduced into the building, the earthquake loads caused the largest values of base shear and moment and therefore controlled the design from a geotechnical aspect.

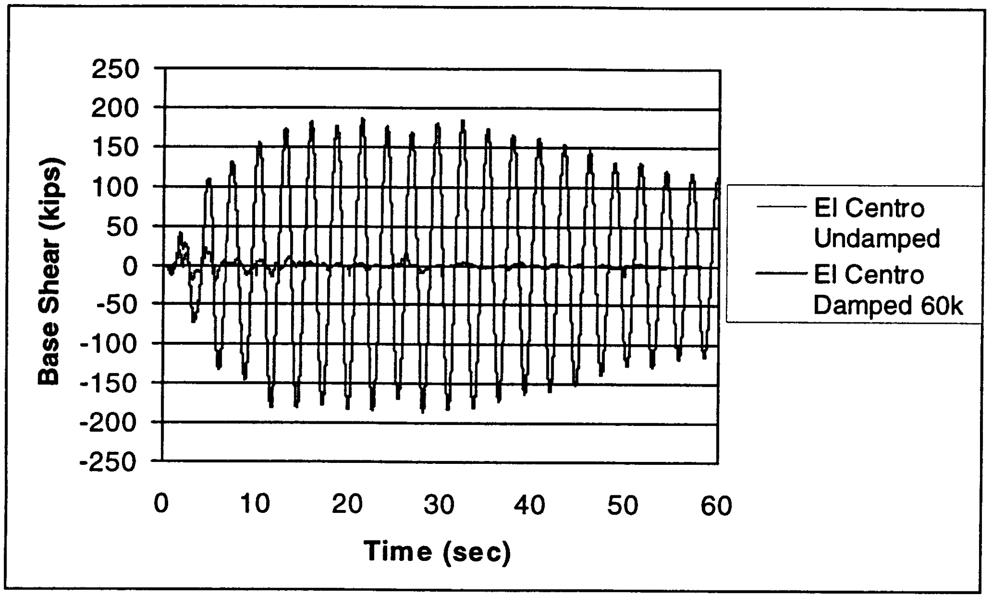

Figure 6-9: Base shear due to El Centro

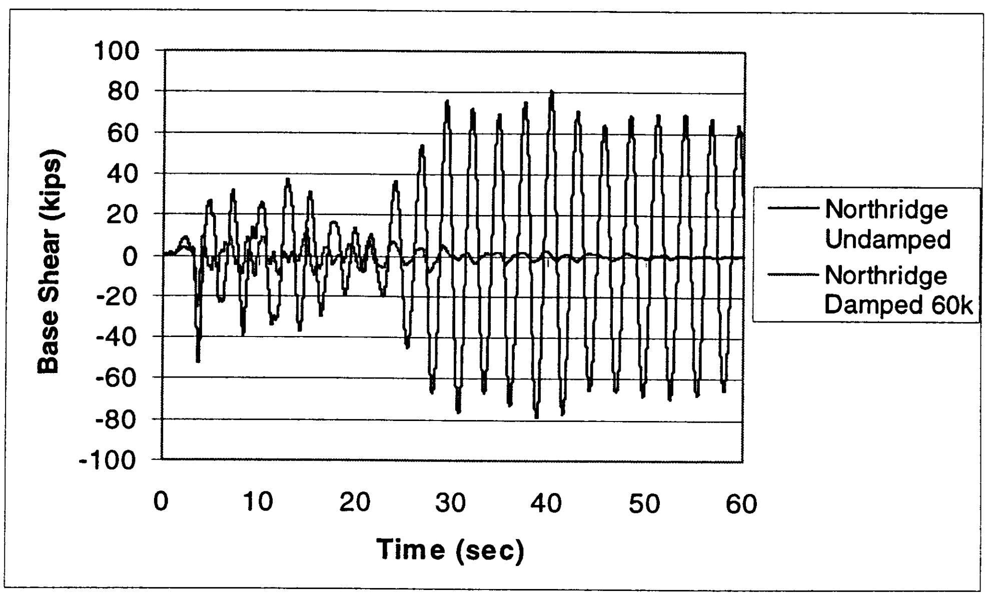

Figure 6-10: Base shear due to Northridge

6.7 *Input Energy*

The energy input into the system was graphed as a function of time for both the damped and undamped cases. See Appendix B for the results. This energy results from the ground excitation due to the seismic loading and for a single degree-of-freedom is given by the following equation:

$$E_I = -m\int_0^t a_g(t)\dot{u}(t)dt \qquad \text{Equation 6-1}$$

The input energy is dependent on the mass of the system, the ground acceleration, and the velocity of the system relative to the ground. Therefore, if the velocity of the structure were decreased, the potentially destructive energy applied to the structure would also be reduced.

The velocity of the system can be described by the equation

$$\dot{u}(t) = \int_0^t a_g(\tau)\cdot e^{-\xi\omega(t-\tau)}\cos[\omega(t-\tau)]d\tau \qquad \text{Equation 6-2}$$

In this equation, ω is the undamped natural frequency and ξ is the damping ratio. If more damping is introduced into the system, thereby increasing the damping ratio, the magnitude of the velocity will be reduced. It will also quickly decay due to the exponential term.

As can be seen by the input energy curves in Appendix B, the amount of energy the structure was required to dissipate was reduced significantly by the addition of fluid viscous dampers. This is important because input energy is converted into three different forms of energy by a system: kinetic energy, strain energy, and dissipated energy. Therefore, without dampers to rapidly dissipate the input energy, it is converted to a combination of kinetic energy and strain energy. Either the motion of the structure is increased or its members are deformed. The kinetic energy should be kept to a minimum because structures usually have motion limits imposed on them by building codes or space restrictions. The strain energy represents the relative deformations of the components of a structure. If a large part of the input energy is converted into strain energy, the possibility of failure of the structural components is increased.

The equations described in this section were taken from chapter two of *Introduction to Motion Based Design* by Jerome J. Connor and Boutros S.A. Klink [7].

7 ADINA Finite Element Analysis

7.1 Model

The finite element program Automatic Dynamic Incremental Nonlinear Analysis, or ADINA, was also implemented for this thesis. ADINA was utilized to examine the behavior of a typical frame of the 5-story section of the suspended structure. The reason this analysis was performed with a second software package was twofold. First, the results could be compared with the previously obtained data from SAP2000. By doing this, an overall estimation of how accurate the results are can be ascertained. The second motivation behind this analysis was to provide an alternate method for this investigation and gain experience with the finite element package.

An illustration of the model is shown in Figure 7-1. As opposed to the SAP2000 model, the frame input into ADINA was only drawn in two dimensions. The reason for this was that all of the members had already been designed. ADINA was only utilized to examine the dynamic behavior of the frame and its response to lateral loading. As before, the x-direction was the focus point for the analysis. Therefore, only a two dimensional model was deemed necessary.

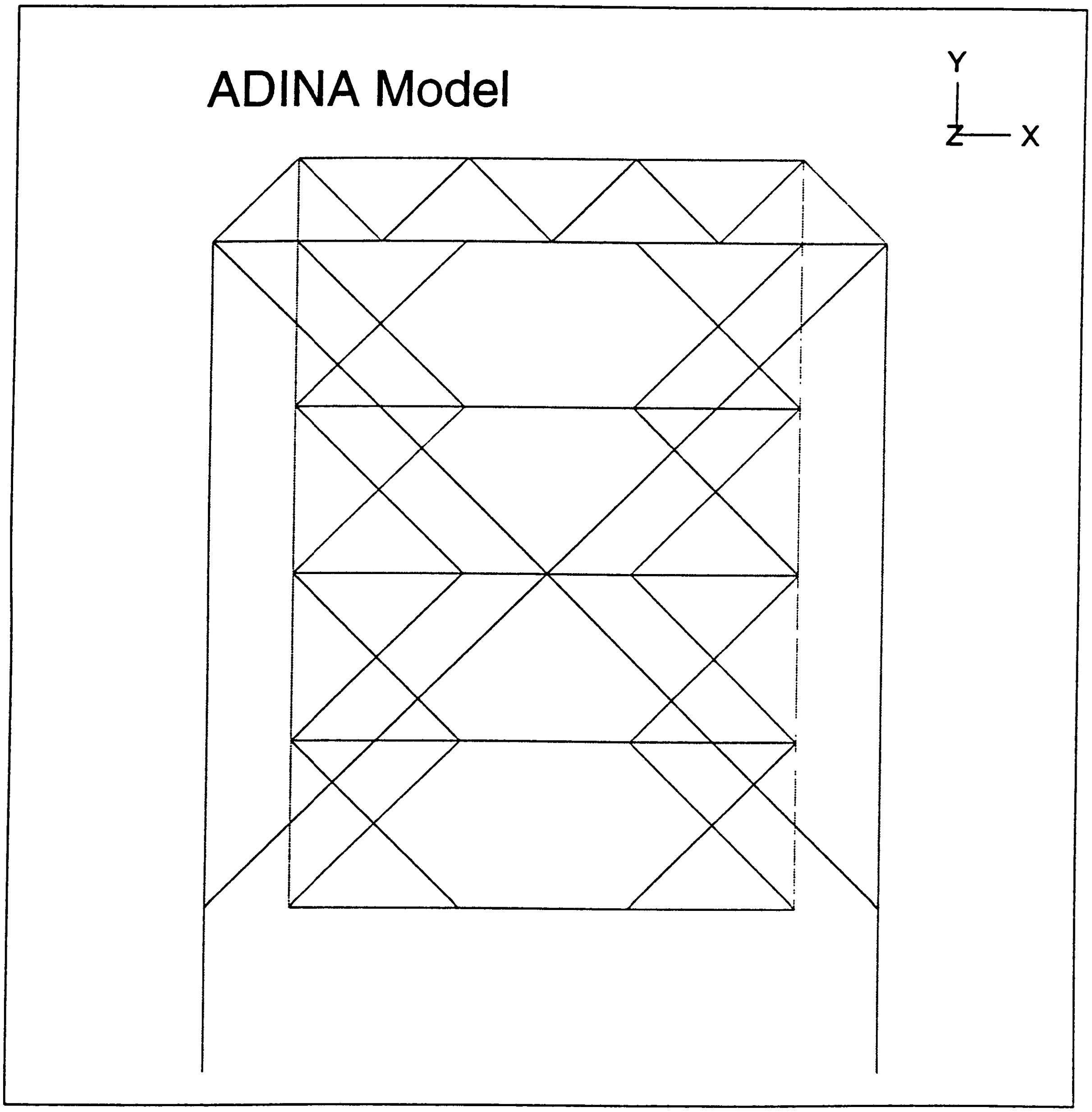

Figure 7-1: ADINA model of structural frame

The previously designed member sizes were input into ADINA in the appropriate format. The girders and truss members were characterized as W-sections. The exterior columns were identified as box sections with a fixed boundary condition at the base. The interior cables, external lateral bracing, and internal lateral bracing were all classified as truss sections. The assumption was made that the double angle sections, which comprise the internal lateral bracing, only work in tension, similar to the cables. This is a reasonable assumption and allows the members to be classified as truss sections with an equivalent cross sectional area. A truss element differs from a beam element in that it does not support moments at its ends.

The forces applied to the girders, by way of the beams, needed to be characterized. The forces can be divided into the dead load due to the self-weight of the slab and beams and the slab design live load of 200 psf. The loads could not simply be added together and applied as point forces at the positions where the beams connect to the girders. This would alter the response of the model to an earthquake load. As was previously mentioned, the force experienced by a structure due to the ground accelerations produced by an earthquake can be classified as inertial forces. The mass of the body acted upon affects the magnitude of these inertial forces. Therefore, if the dead loads were applied to the girders as point forces the mass of the model would be less than the actual mass of the structure. While the deflected shape due to gravity would still be correct, this would lead to an inaccurately lower response of the model to seismic loading. To account for this, the weight of the slab and the beams was applied to the girders as concentrated masses at the correct locations.

7.2 Analysis Methodology

Once the model was created in ADINA, the first step was to check the deflected shape of the structure due to gravity. See Appendix C for an illustration of the deformed shape of the model due to the applied concentrated masses and the live load point forces. The structure appears to behave in the expected manner to the applied gravity loads. The

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The bending moment diagram from ADINA displays similar inconsistencies. The SAP2000 bending moment diagram reveals a continuous curve with a maximum value at the center of each girder. Meanwhile, the ADINA diagram again oscillates between positive and negative values. Based on the loading and preliminary analyses, the decision was made to use the SAP2000 model and results as the correct representation of the response of the suspended structure. Therefore, the previous seismic loading analysis performed in SAP2000 is still valid.

Although many attempts were made to recreate and correct the model of the suspended structure in ADINA, the problems could not be resolved. At first, the concentrated masses were eliminated. The live load was applied and the deformed shape was examined again. The shapes of the shear and bending moment diagrams were identical to what was previously stated.

A simply supported beam with concentrated point loads was then analyzed in ADINA to confirm the preliminary analysis and make sure the ADINA results were being correctly interpreted. The shear force and bending moment diagrams proved to agree with the expected results and those obtained from SAP2000. This confirms that the ADINA results are being evaluated correctly.

The internal bracing was eliminated from the model in a final effort to determine what the cause of the error was. With the bracing removed, the model could not be compiled. The stiffness matrix had pivot terms equal to zero. This results when the model is not properly supported. The interior cables do not provide the structure with any stiffness, resulting in the need for the internal bracing. Therefore, there was no way to tell if the bracing was the cause of the problems with the model.

The results of this investigation are a good example of why preliminary analyses should always be performed before a computer program is utilized. An engineer must always use his best judgment when deciding whether to accept a computer's solution as an accurate representation of the physical problem. A computer should never be thought of as a "black box" that always produces a reasonable output. Analysis software is only as good as the information that is initially input by the user. Inaccurate or incorrect input will yield bad results for the desired problem or results for a completely different problem altogether. Any kind of analysis should be performed by a person who is knowledgeable on the subject and can interpret results as well as utilize the software correctly.

7.3 Dynamic Analysis Results

Although the problems with the ADINA model could not be detected, a preliminary dynamic analysis was performed. The mode shapes and natural periods of the suspended structure were determined for comparison with the previously obtained results. See Figure 7-2 for a diagram of the first mode shape. See Appendix C for pictures of the other mode shapes.

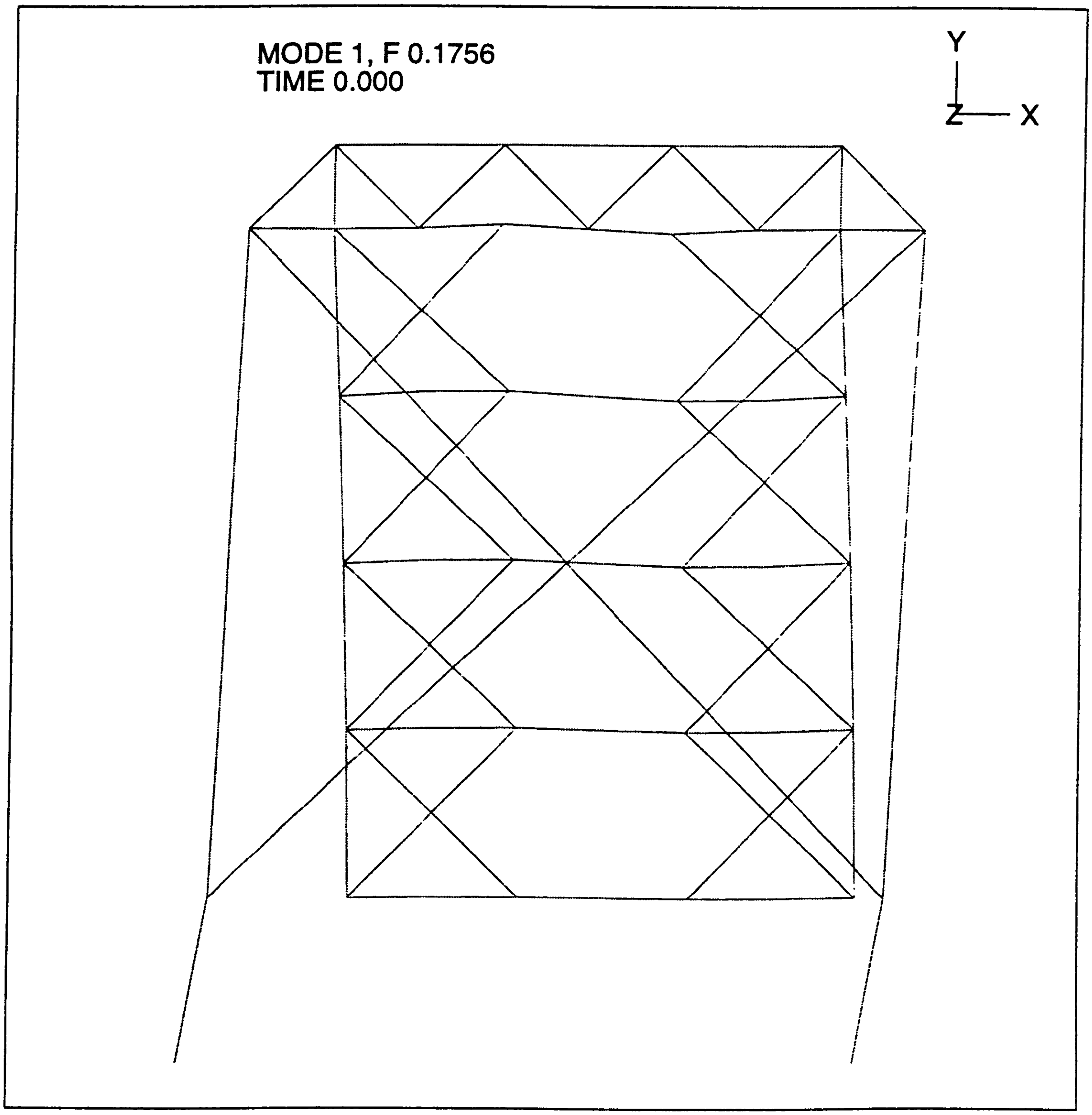

Figure 7-2: First mode shape

The mode shapes obtained from ADINA are comparable in appearance to the ones obtained earlier from SAP2000. The first mode displays the case where the entire frame sways to the side. In this case, the suspended floors move together. In the second mode, the suspended structure dominates the building's lateral movement and the floors are 180 out of phase. The shape of the third mode corresponds to the deflected shape of the structure due to gravity loading.

The natural periods of the structure, corresponding to the modes mentioned above, were also calculated. See Table 7-1 for a comparison between the results from SAP2000 and ADINA. As one can see, the values produced by ADINA are slightly higher than the periods found by SAP2000.

Mode Number	*SAP2000 (seconds)*	*ADINA (seconds)*
1	2.8335	5.6948
2	0.5532	1.3314
3	0.2699	0.9141

Table 7-1: Natural periods of suspended structure

The disparity between the results displayed in Table 7-1 can be attributed to the fact that the ADINA model has some flaws. As was shown earlier, the shear and bending moment diagrams are inaccurate for the girders. These errors were never located and corrected. Therefore, it is reasonable to assume that the results from ADINA have error associated with them. It is encouraging to note that the SAP2000 results are much closer to the expected values.

The building was originally assumed to have a natural period of about one-second. The first natural period from SAP2000 is slightly higher than this, but this difference can be explained. The original assumption was based upon the number of floors of the building. This assumption is usually made for floor heights in the range of ten to fifteen feet. The suspended structure has twenty feet between consecutive levels. The exterior columns also have an effective multiplier of two due to the fixed-free boundary conditions. These two facts taken together cause the period of a structure to increase.

8 Conclusion

8.1 Damping Recommendation

A judgment had to be made as to how much damping is sufficient. A value of L/360 was used for the deflection limit at the top of the structure. This is a common value for maximum lateral displacements of buildings. The deflection limit was therefore set at 3.3 inches. The maximum deflection at the top of the exterior columns was reduced to 4.2 inches by the first damping scheme. When the damping force was increased, the maximum deflection was further reduced to 2.8 inches. Based on the L/360 criterion, it seems that the second scheme, with two 60 kip dampers in a typical frame of the building, yields the desired results.

Damping Cost

An essential part of the decision on how much damping to incorporate into a building includes factoring in the cost of the dampers. Taylor Devices markets 150 kip dampers for $7,000 each and 300 kip dampers for $13,000 each [2]. The first damping scheme would cost $42,000 for six 150 kip dampers. The second scheme would cost $78,000 for six 300 kip dampers or $84,000 for twelve 150 kip dampers. Based upon the deflection results and the cost of the various schemes, the best alternative seems to be to install six 300 kip dampers in the structure at a cost of $78,000.

This only represents the damping required for the building to resist motion in the x-direction. The assumption was made that the building would react similarly in the x- and y-directions due to its symmetry. Therefore, for the purposes of the cost estimate, it will be assumed that the same size and number of dampers are required in the y-direction as well. The total cost of the twelve, 300 kip, fluid viscous dampers would then be $156,000. Two pairs will be installed at each end of the building and in the frame that separates the 5-story section from the 3-story section.

8.2 Internal Lateral Bracing Cost

The cost of the internal steel bracing members was also calculated and included in the final cost of the lateral load system for the building. The cost was determined by computing the total volume of steel required by the braces. The standard price of \$390,000 per cubic meter of steel was then utilized. The resulting cost of twenty steel braces is \$118,000. See Appendix A for the required calculations.

8.3 Final Recommendations

The final recommendations for optimal damping for the suspended structure that was analyzed for the purpose of this thesis are as follows. Fluid viscous dampers should be incorporated in the structure to combat lateral loading due to earthquakes. Twelve 300-kip dampers are required for both the x- and y-directions. The dampers will be installed at an angle of 45° to the ground and attached to the base of the suspended structure where the motion is largest. The dampers will be installed in the two end frames of the building and the frame between the 5-story section and the 3-story section.

The braces are located in the same frames as the fluid dampers. They will be concealed by enclosing them in the walls surrounding the elevator shafts. Twenty steel braces were required for the structure. The total cost for the lateral system is about \$275,000. While this may seem like a large amount of money to spend initially, the benefits outweigh the cost. As was previously shown, fluid dampers absorb potentially destructive energy input on a system. When incorporated into the lateral load bearing system of a building, they will prevent damage and possibly irreparable failure from occurring. The cost associated with repairing a damaged structure is usually too high to be worthwhile. Therefore, buildings end up being torn down and rebuilt. By simply investing some time and money initially, this scenario can be avoided in the future.

9 References

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12. *Commonwealth of Massachusetts State Building Code.* The Office of the Massachusetts Secretary of State. Sixth Edition. 1997.

9.2 Software Used

AutoCAD v.13

SAP2000 Nonlinear Version

SAP2000 Educational Version

ADINA Finite Element Software

Matlab Release 5.1

Microsoft Excel 97

10 Appendix

APPENDICES

A. Internal Lateral Bracing Design

Earthquake Loads

Aa	Av	S	R	Cd	Fl. Area	Rf. Area	Dead
					sf	sf	kips
0.15	0.12	2.0	7	4	15000	15000	1050

W	Hn	Ca	Ct	Ta	T	Cs
kips	feet			sec	sec	
4200	110	1.6	0.02	0.68	1.09	0.03891928

V	k	h1	h2	h3	h4	h5	w1	w2	w3	w4
kips		feet	feet	feet	feet	feet	kips	kips	kips	kips
163.461	1.57	20	40	60	80	100	1050	1050	1050	1050

Cv1	Cv2	Cv3	Cv4	Cv5	F1	F2	F3	F4	F5
					kips	kips	kips	kips	kips
0.05436	0.161398	0.305043	0.479198	0	8.9	26.4	49.9	78.3	0.0

V1	V2	V3	V4	V5	t	M1	M2	M3	M4	M5
kips	kips	kips	kips	kips		kips*ft	kips*ft	kips*ft	kips*ft	kips*ft
163.5	154.6	128.2	78.3	0.0	1.0	7222.0	4130.5	1566.6	0.0	0.0

Bracing Design Calculations

Dead:	beams	5 psf	
	girders	16 psf	
	slab	34 psf	
	partitions	10 psf	
	mechanical	5 psf	
	TOTAL:		70 psf

Floor Area: (250 ft)(60 ft) = 15,000 ft^2

Total Floor Weight = (4 floors)(15,000 ft^2/floor)(70 psf) = 4200 kips

$$C_s = \frac{1.2 A_v S}{R T^{2/3}}$$

$T_a = C_T h_n^{2/3}$

$T < C_a T_a$

$V_{base} = C_s W$ $\qquad$ $V_{base} = C_s W = (0.0389)(4200 \text{ kips}) = 163.5 \text{ kips}$

$$\frac{163.5 kips}{2 braces} = 81.75 kips / brace \qquad 81.75 kips\left[\frac{28.3 ft}{20 ft}\right] = 115.7 kips$$

$$\frac{115.7 kips}{50 ksi} = 2.31 in^2$$

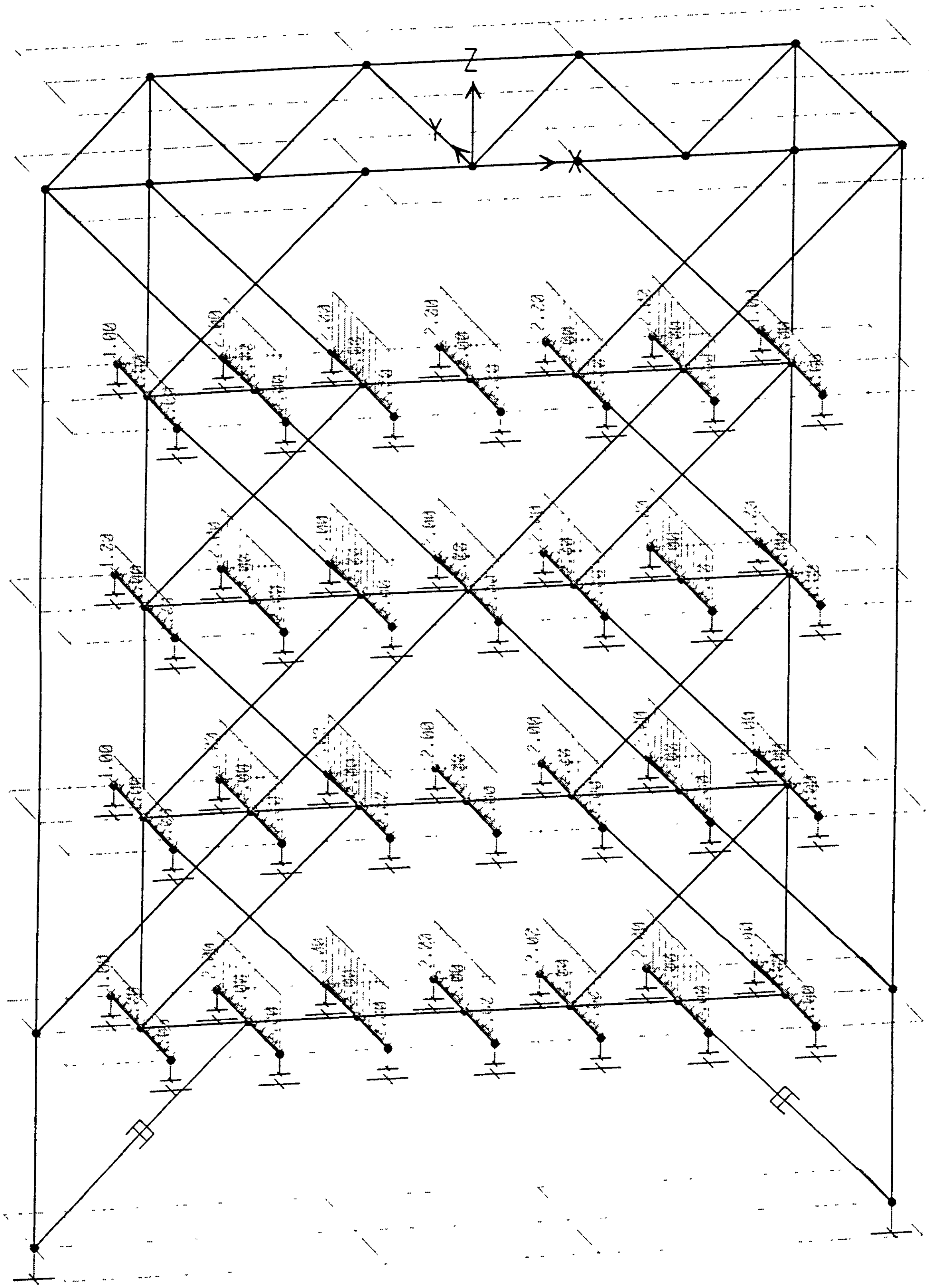
Use two L2x2x $^3/_8$ members with area of 2.72 in^2

Total area of steel required = (20 braces)(2.72 in^2)(28.3 ft)$(0.083 \text{ ft/in})^2$ = 10.7 ft^3

Conversion: (10.7 ft^3)$(0.3048 \text{ m/ft})^3$ = 0.303 m^3

Cost of bracing: (0.303 m^3)($390,000 per m^3) = $118,068

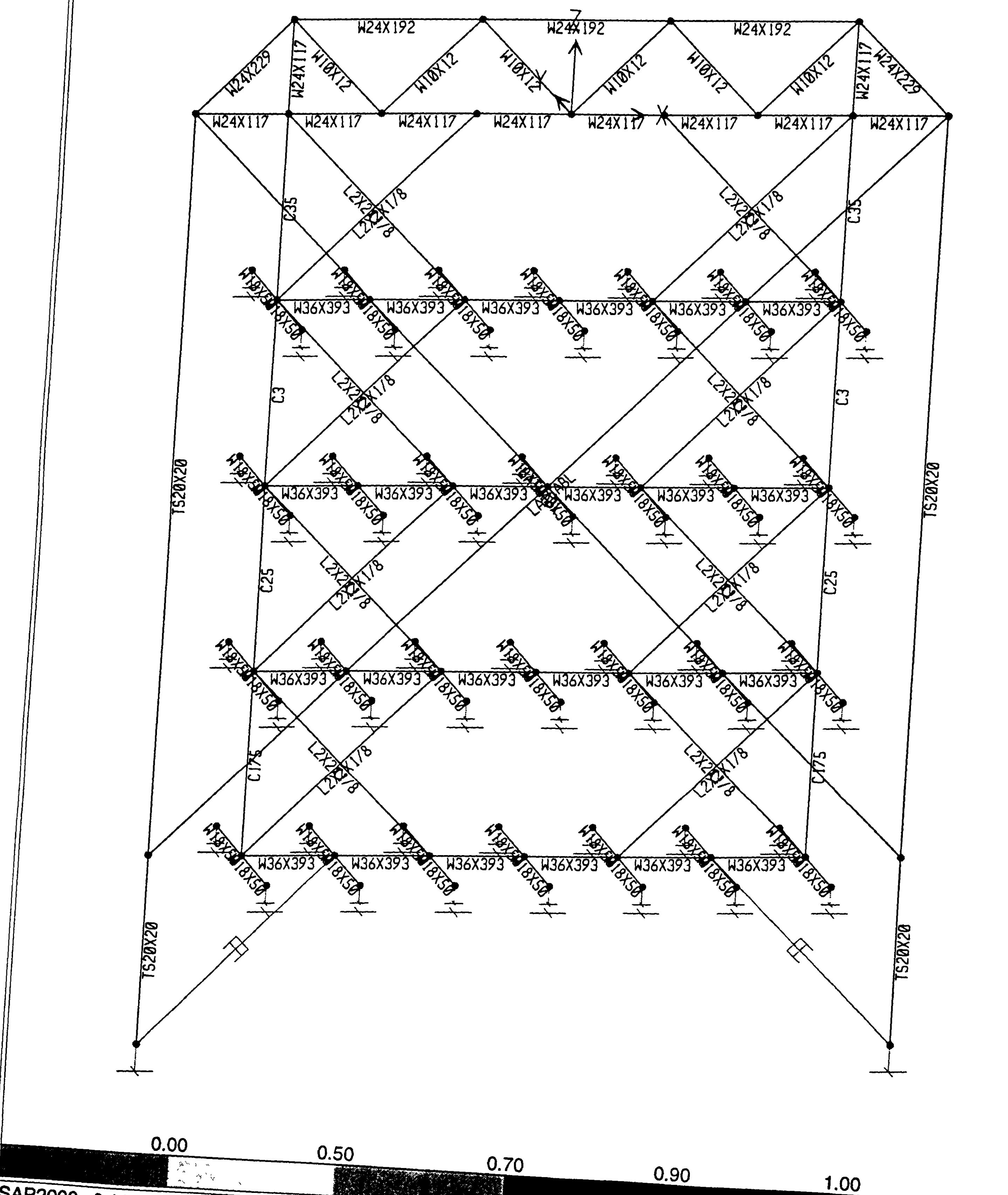
B. SAP2000 Output



SAP2000 v6.11 - File:frame3d1a - Frame Span Loads (LIVE) - Kip-ft Units

SAP2000

April 9,1998 16:30



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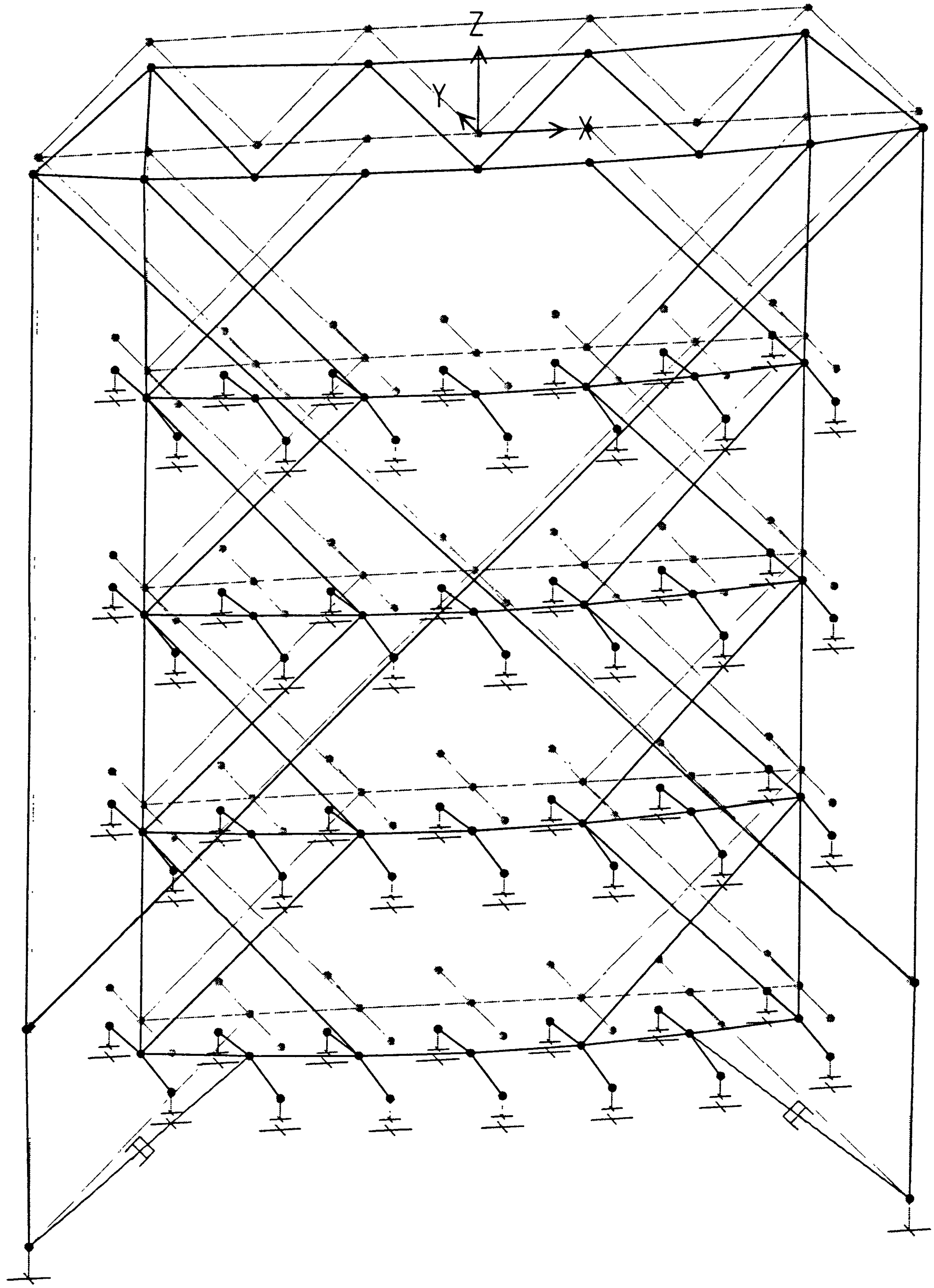
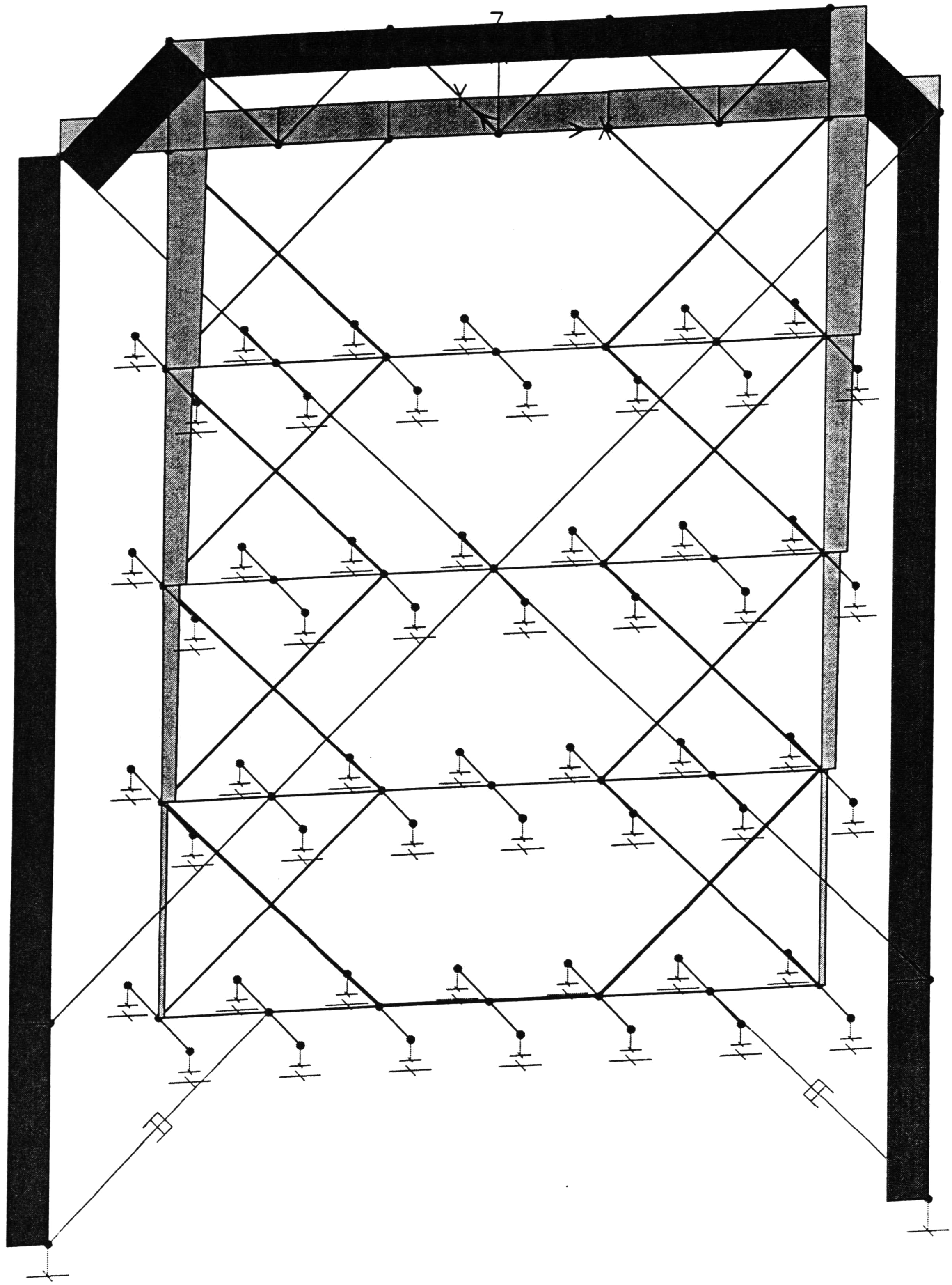
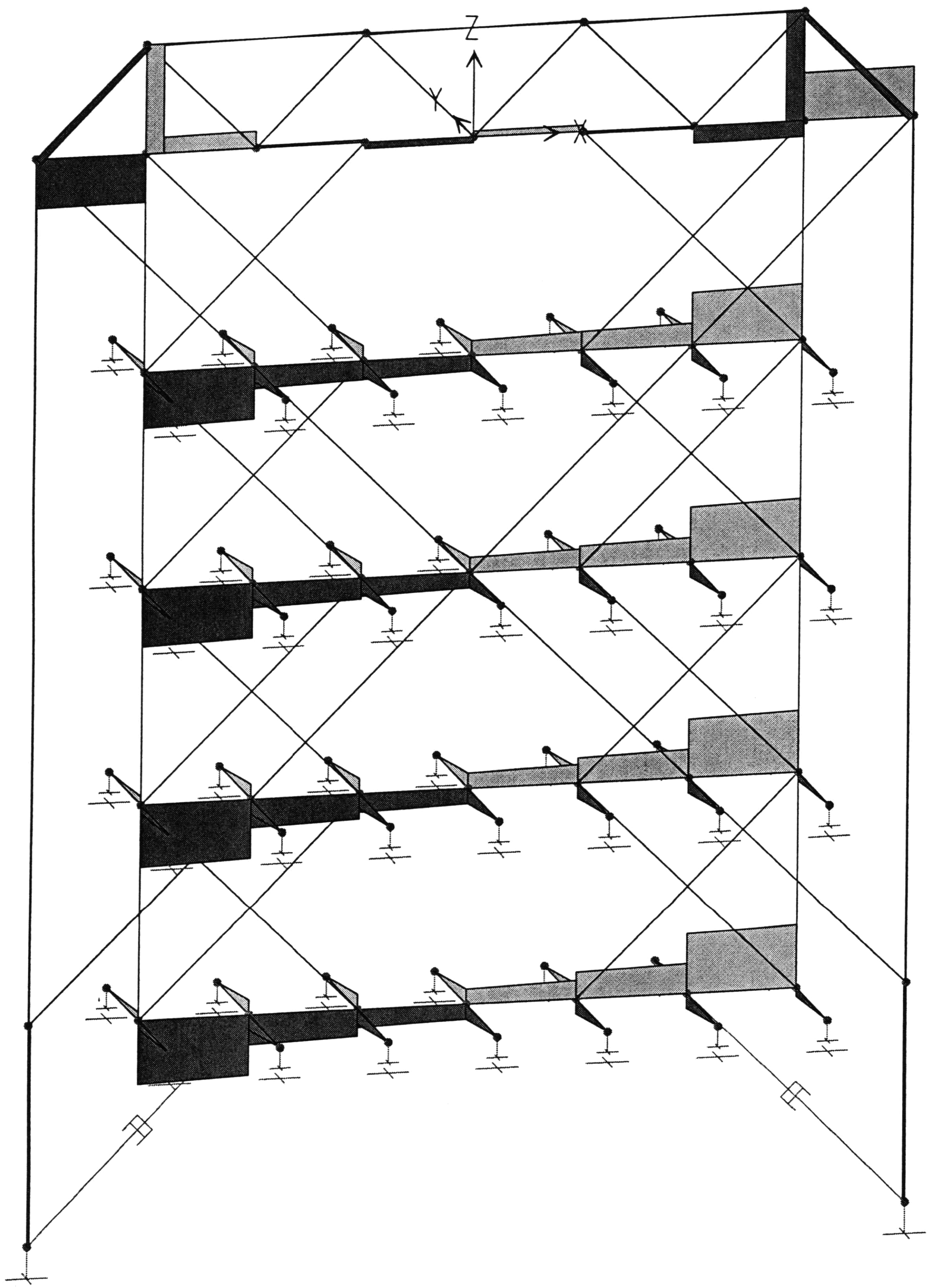


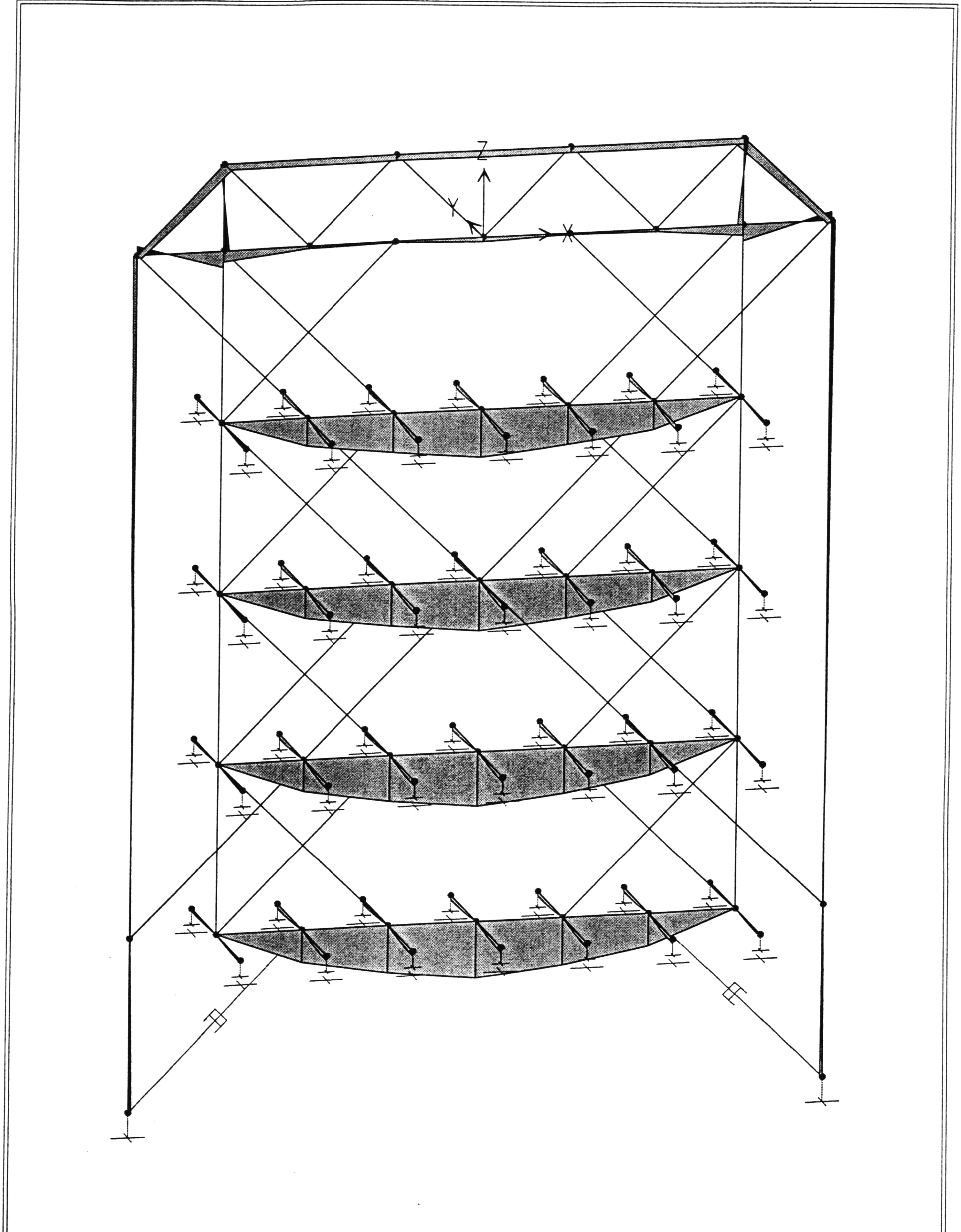
SAP2000 v6.11 - File:frame3d1a - Deformed Shape (COMB2) - Kip-ft Units



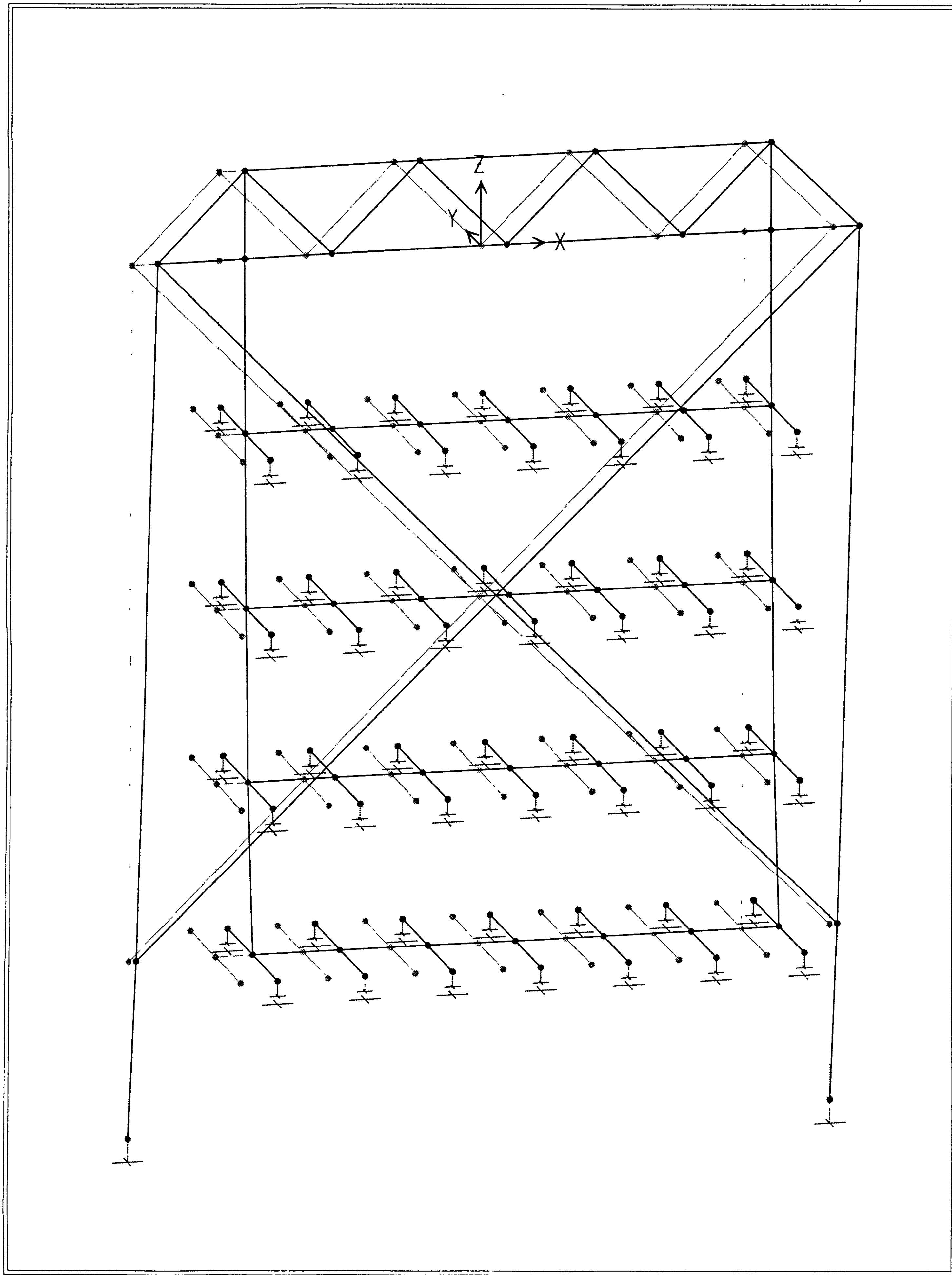
SAP2000 v6.11 - File:frame3d1a - Axial Force Diagram (COMB2) - Kip-ft Units



SAP2000 v6.11 - File:frame3d1a - Shear Force 2-2 Diagram (COMB2) - Kip-ft Units

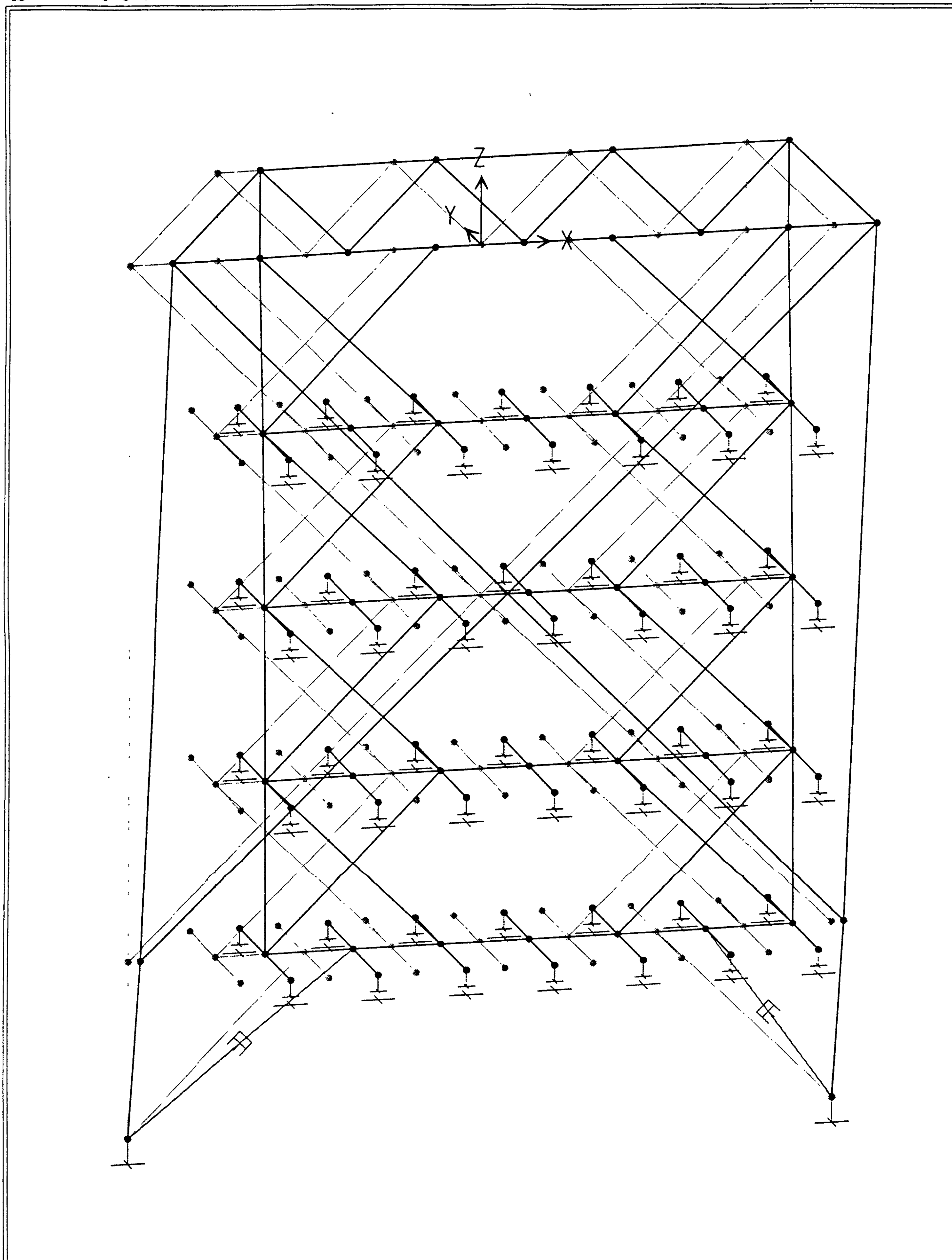


SAP2000 v6.11 - File:frame3d1a - Moment 3-3 Diagram (COMB2) - Kip-ft Units

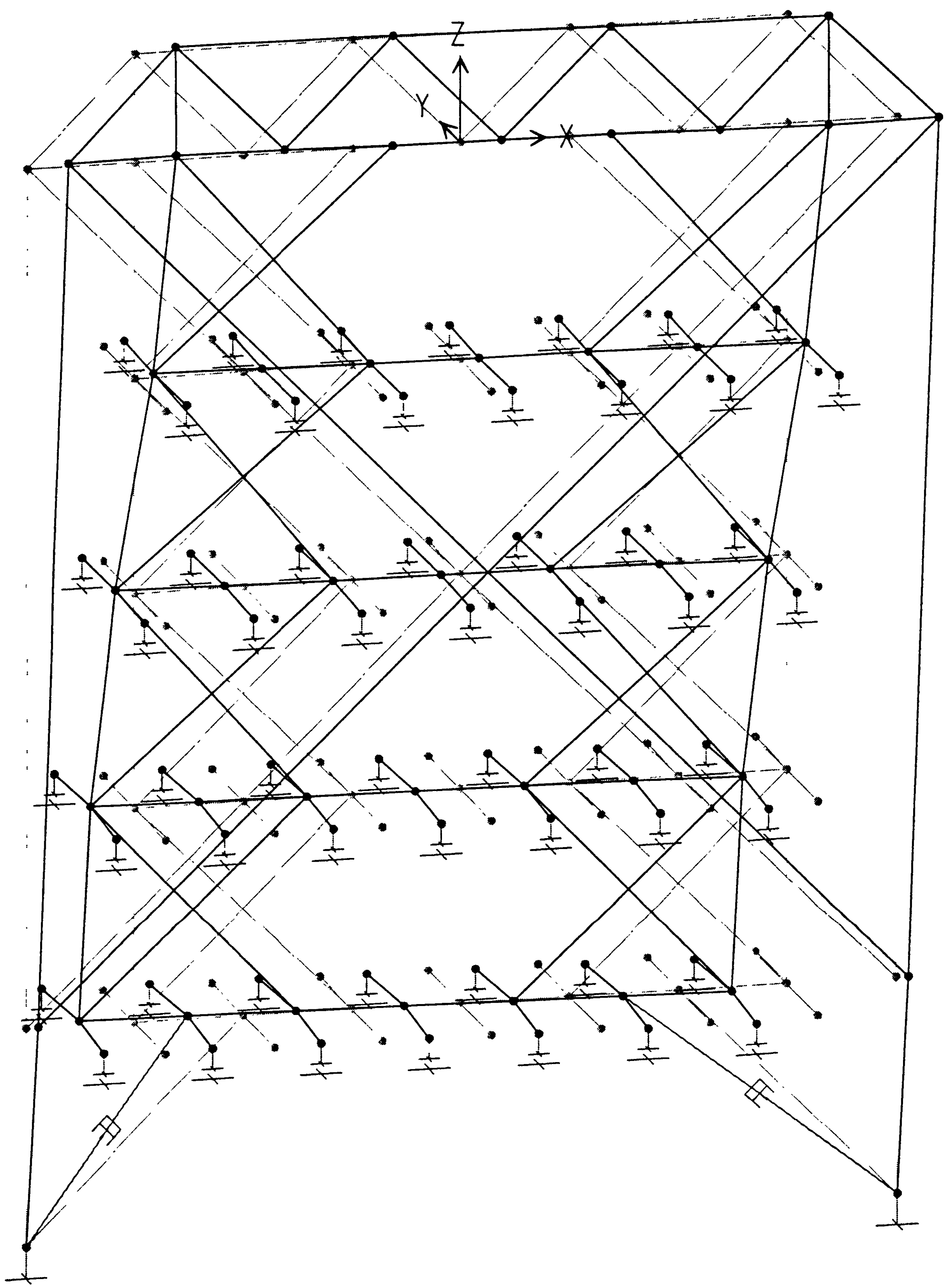


SAP2000 v6.11 - File:frame3 - Deformed Shape (SPEC2) - Kip-ft Units

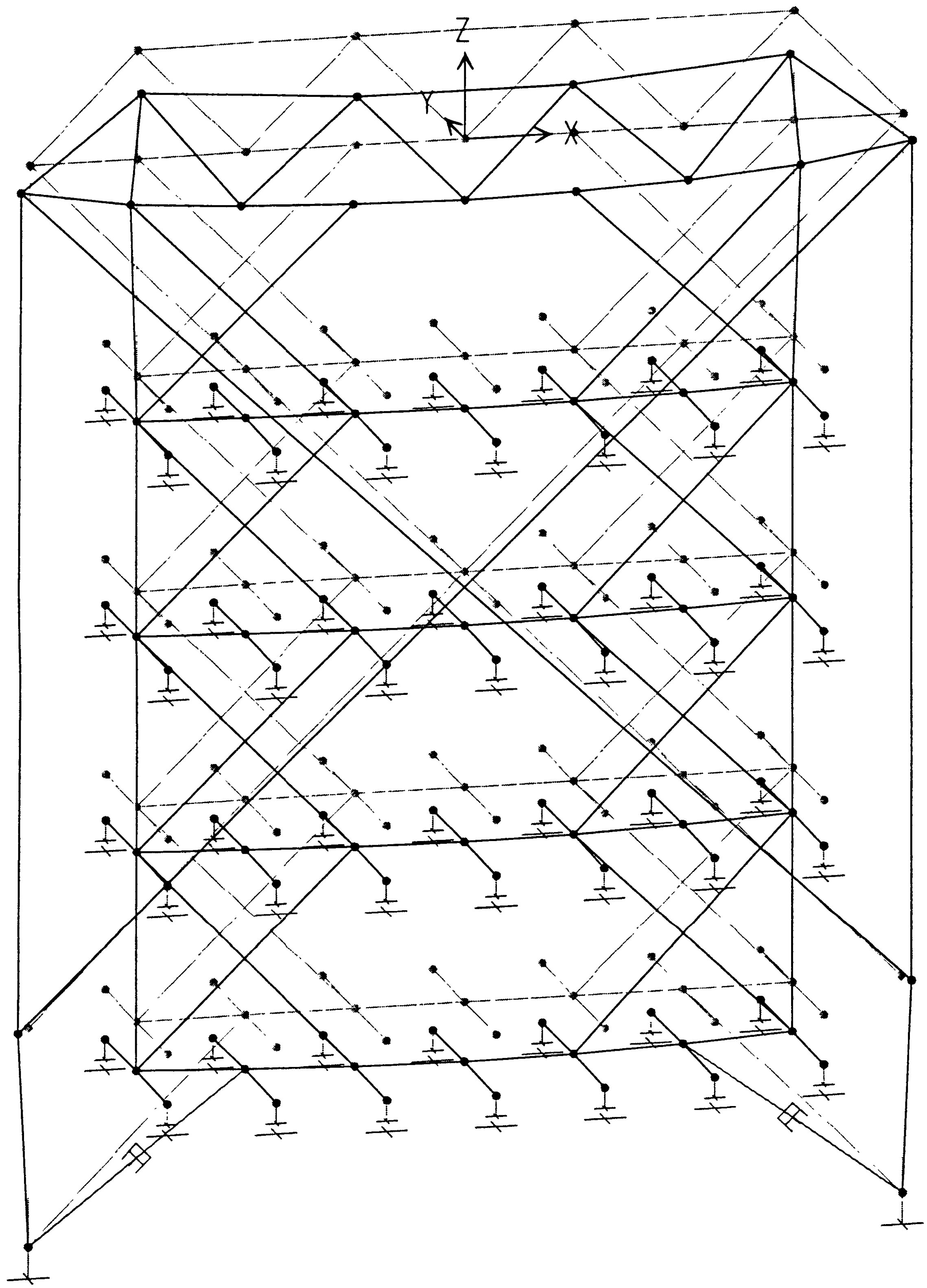
SAP2000 April 9,1998 16:28



SAP2000 v6.11 - File:frame3d1a - Mode 1 Period 2.8335 seconds - Kip-ft Units



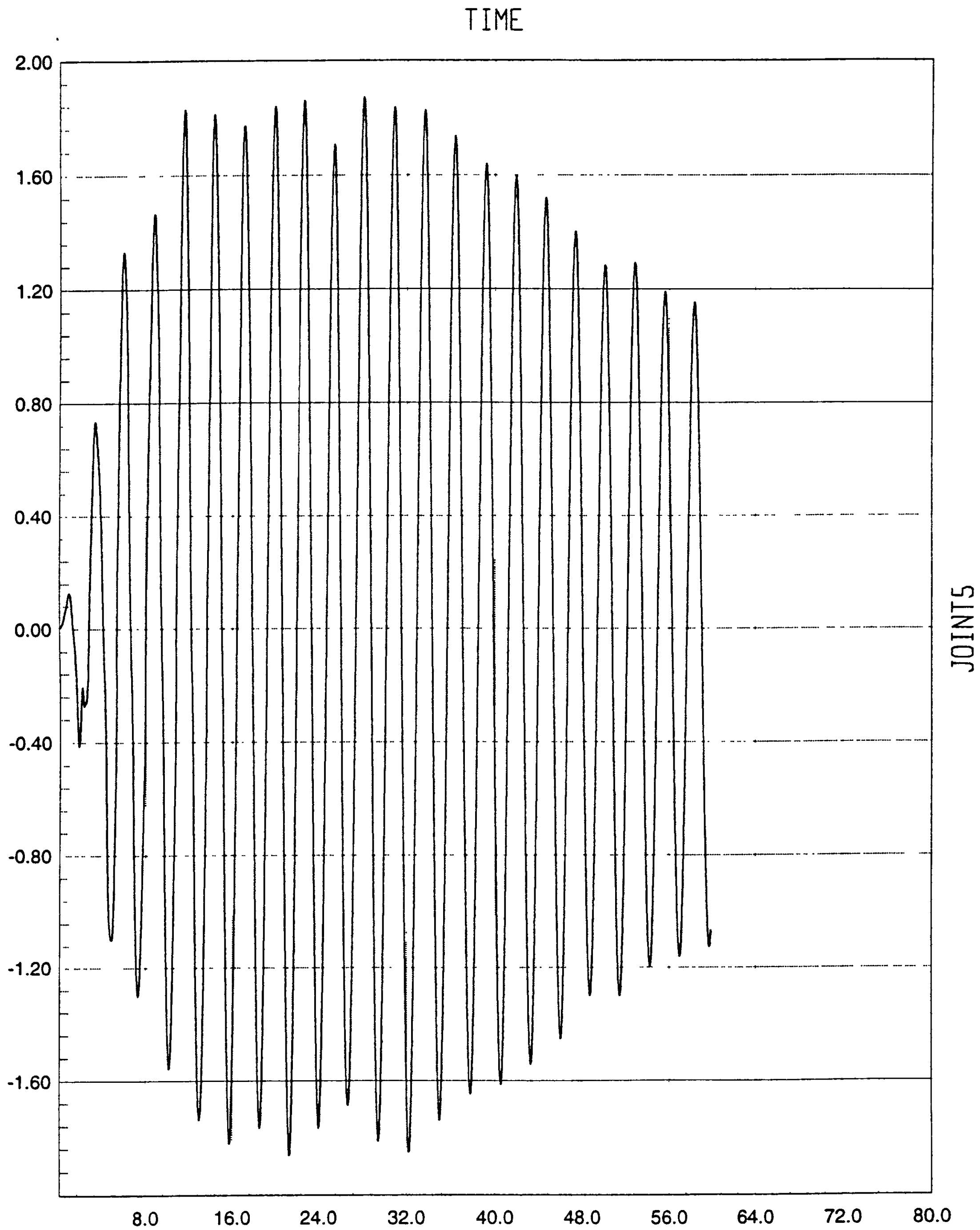
SAP2000 v6.11 - File:frame3d1a - Mode 2 Period 0.5532 seconds - Kip-ft Units



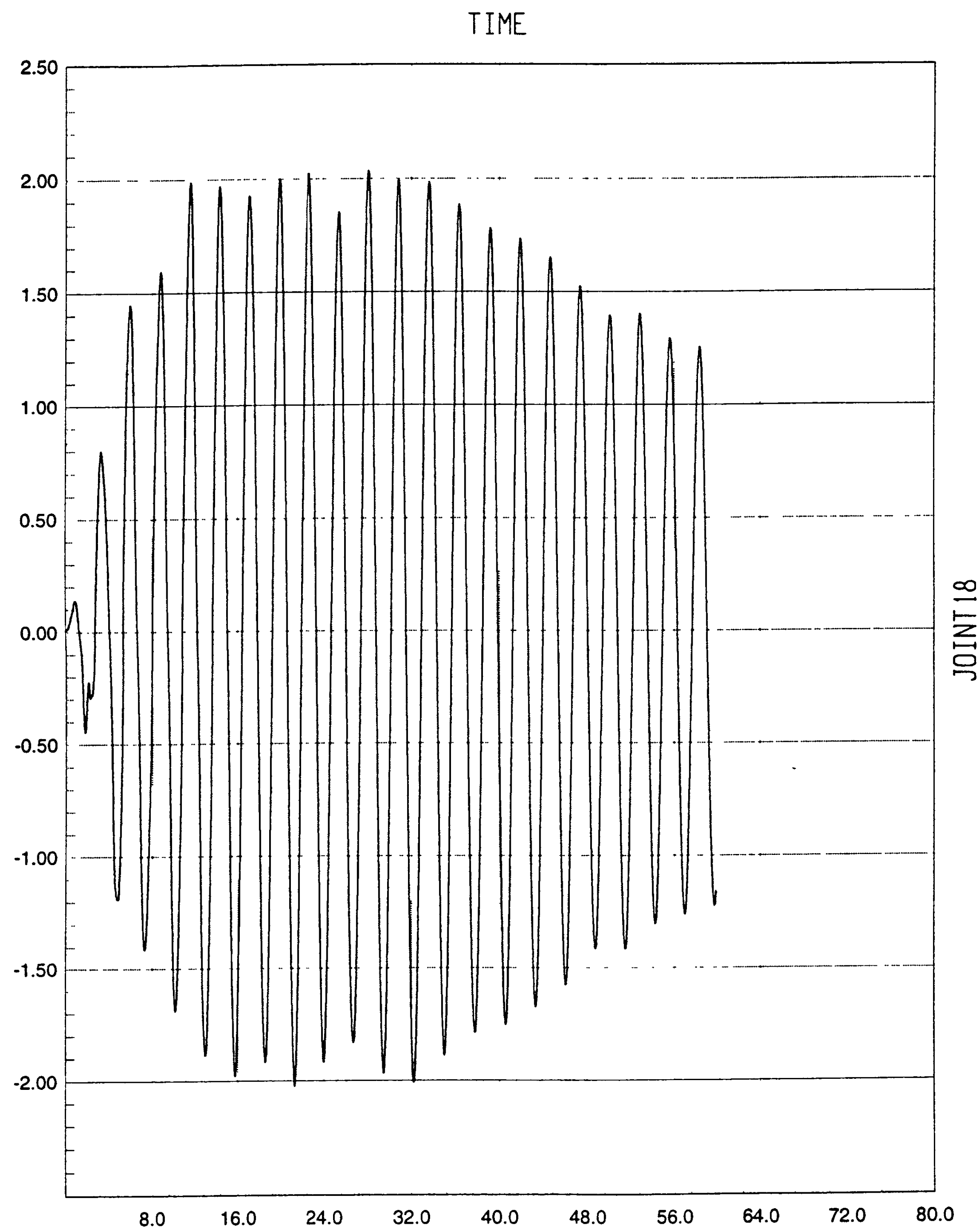
SAP2000 v6.11 - File:frame3d1a - Mode 4 Period 0.2699 seconds - Kip-ft Units

SAP2000

Time History Functions April 9,1998 16:39

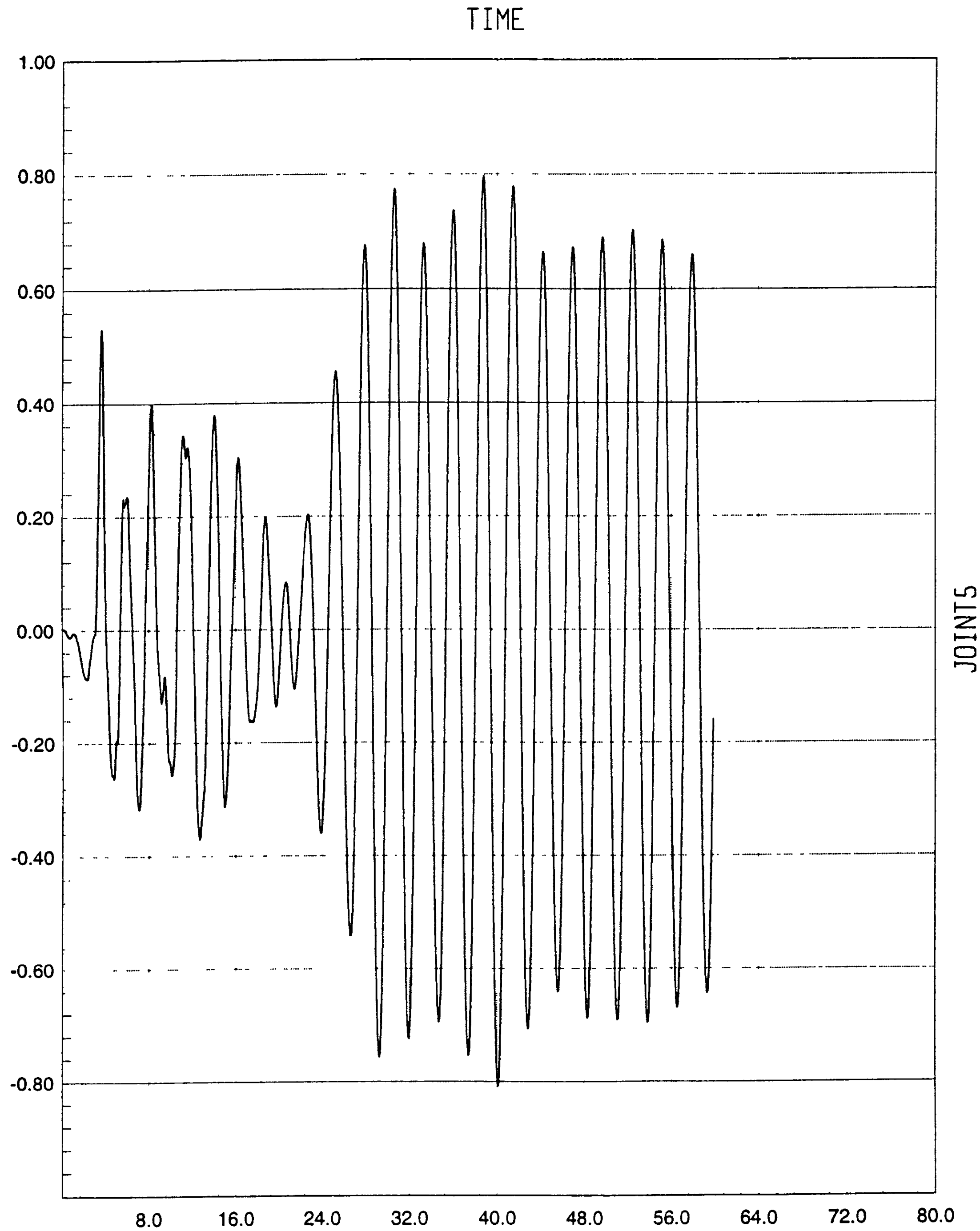


SAP2000 v6.11 - File:frame3 - Kip-ft Units
Joint5: Joint 5 Displacement UX Vs Time
Min is -1.875e+00 at 2.1340e+01 Max is 1.871e+00 at 2.8160e+01



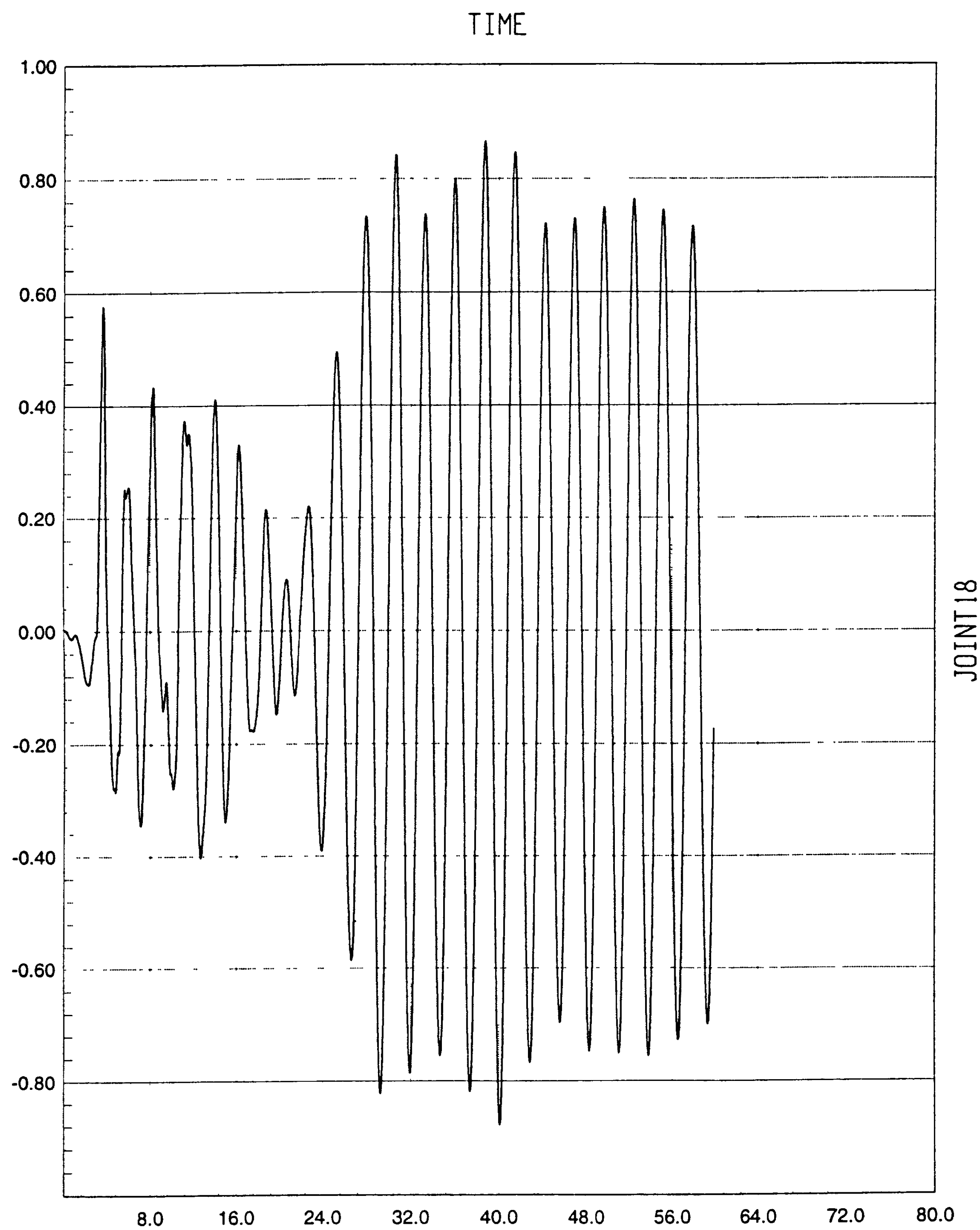
SAP2000 v6.11 - File:frame3 - Kip-ft Units
Joint18: Joint 18 Displacement UX Vs Time
Min is -2.036e+00 at 2.1340e+01 Max is 2.032e+00 at 2.8160e+01

SAP2000 Time History Functions April 9,1998 16:39

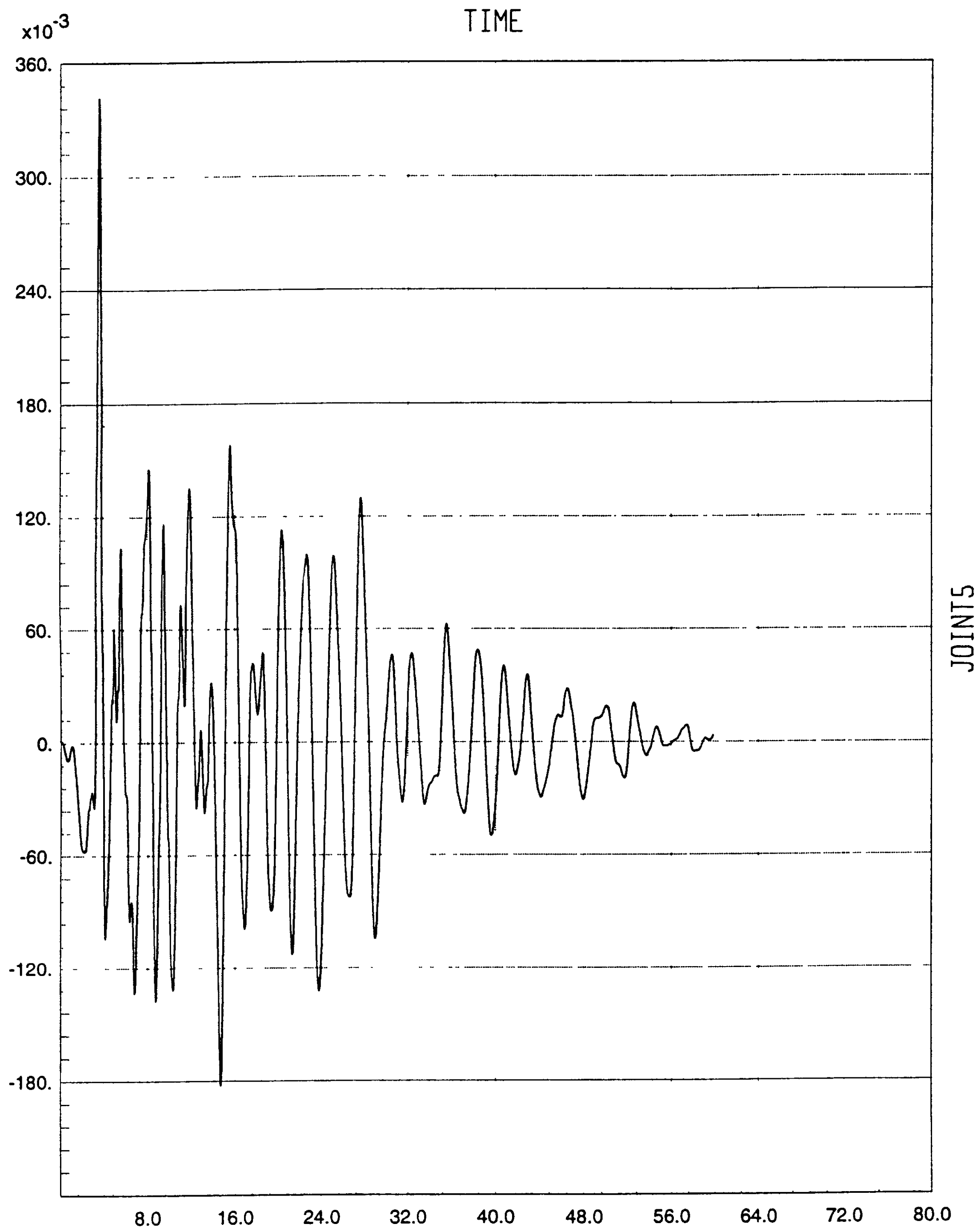


SAP2000 v6.11 - File:frame3 - Kip-ft Units
Joint5: Joint 5 Displacement UX Vs Time
Min is -8.133e-01 at 4.0160e+01 Max is 7.954e-01 at 3.8820e+01

SAP2000 Time History Functions April 9,1998 16:39

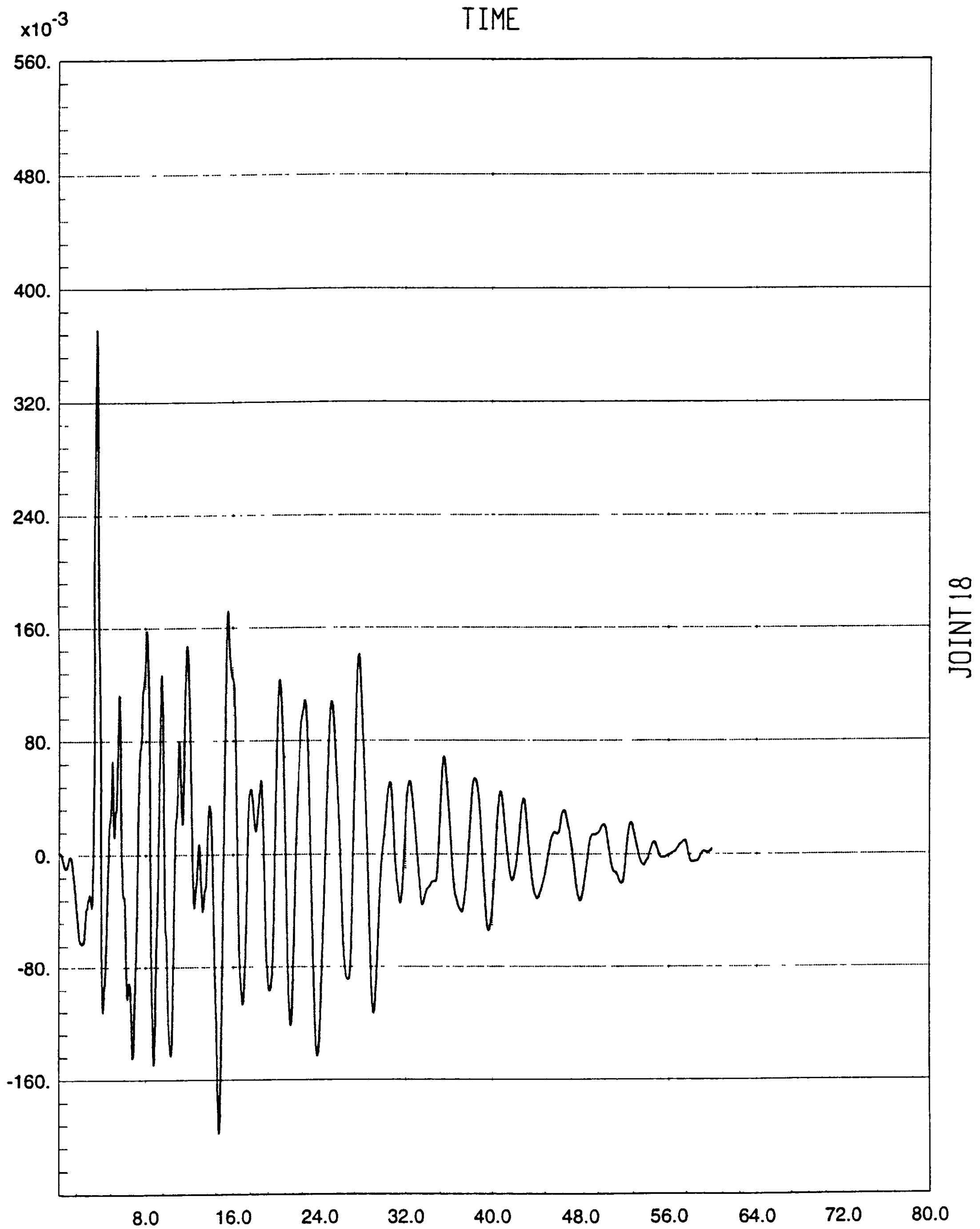


SAP2000 v6.11 - File:frame3 - Kip-ft Units
Joint18: Joint 18 Displacement UX Vs Time
Min is -8.832e-01 at 4.0160e+01 Max is 8.637e-01 at 3.8820e+01



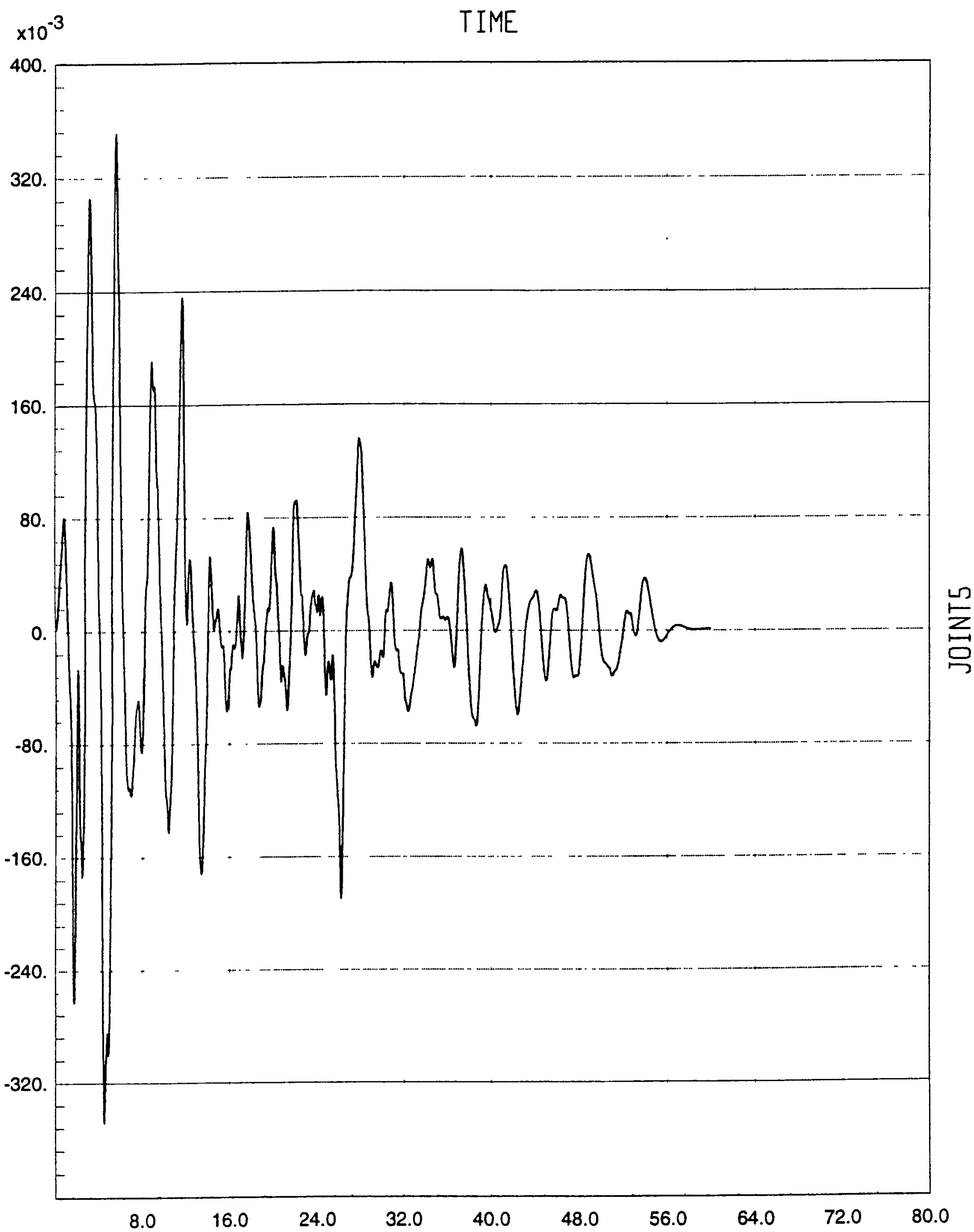
SAP2000 v6.11 - File:frame3d2 - Kip-ft Units
Joint5: Joint 5 Displacement UX Vs Time
Min is -1.839e-01 at 1.4740e+01 Max is 3.412e-01 at 3.5800e+00

SAP2000 Time History Functions April 9,1998 16:36



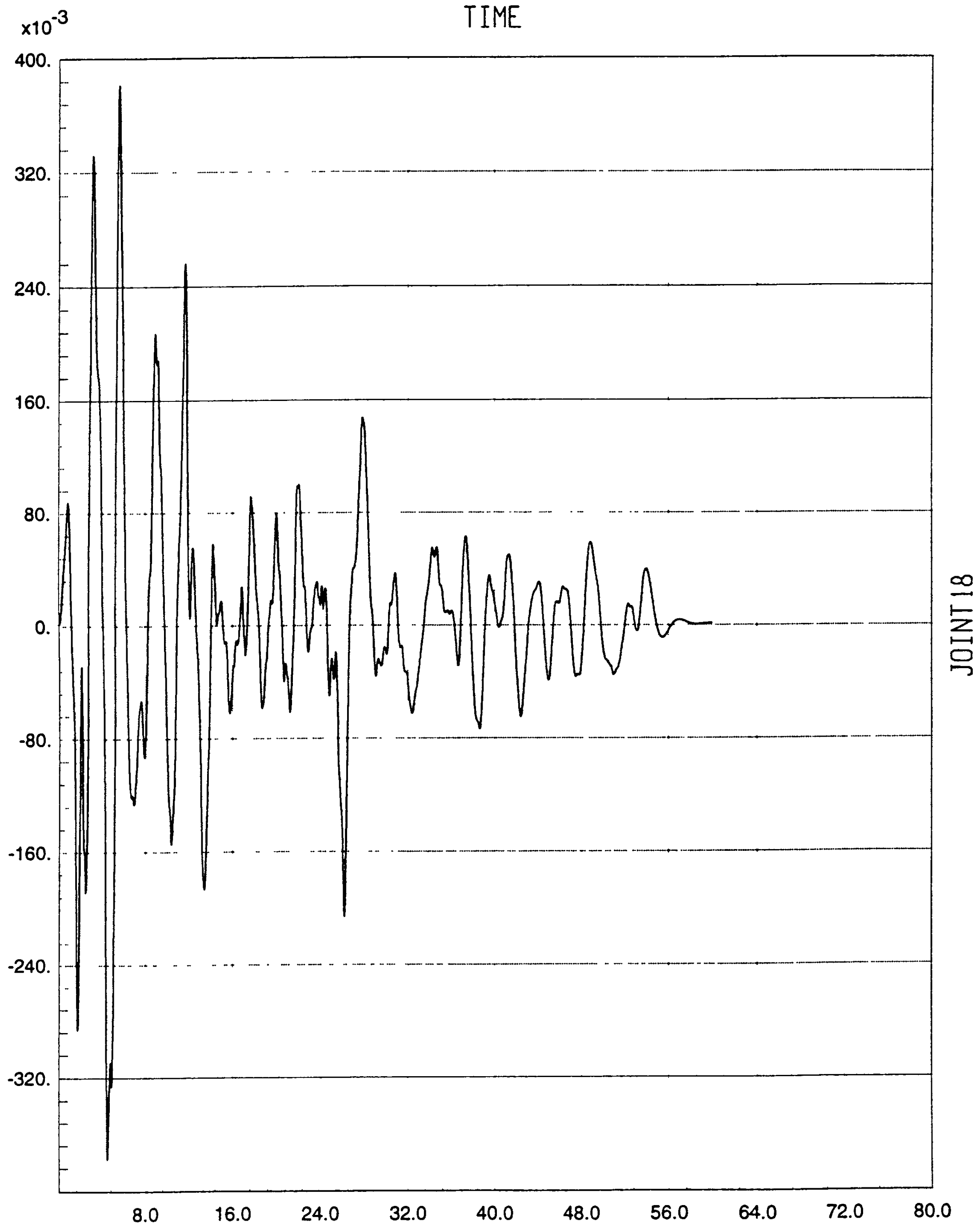
SAP2000 v6.11 - File:frame3d2 - Kip-ft Units
Joint18: Joint 18 Displacement UX Vs Time
Min is -1.997e-01 at 1.4740e+01 Max is 3.705e-01 at 3.5800e+00

SAP2000 Time History Functions April 9,1998 16:36



SAP2000 v6.11 - File:frame3d2 - Kip-ft Units
Joint5: Joint 5 Displacement UX Vs Time
Min is -3.500e-01 at 4.5400e+00 Max is 3.506e-01 at 5.6400e+00

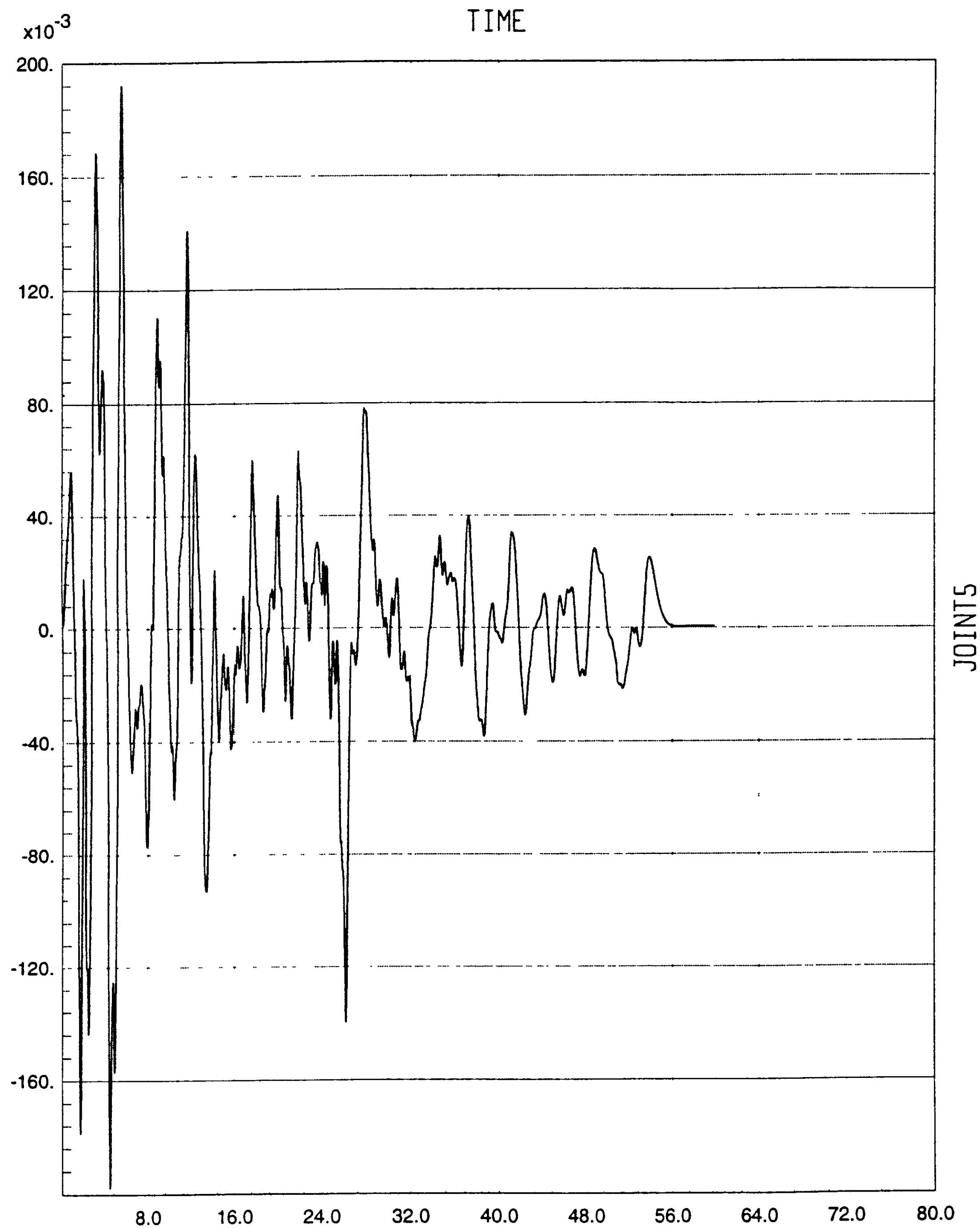
SAP2000 Time History Functions April 9,1998 16:36



SAP2000 v6.11 - File:frame3d2 - Kip-ft Units
Joint18: Joint 18 Displacement UX Vs Time
Min is -3.801e-01 at 4.5400e+00 Max is 3.807e-01 at 5.6400e+00

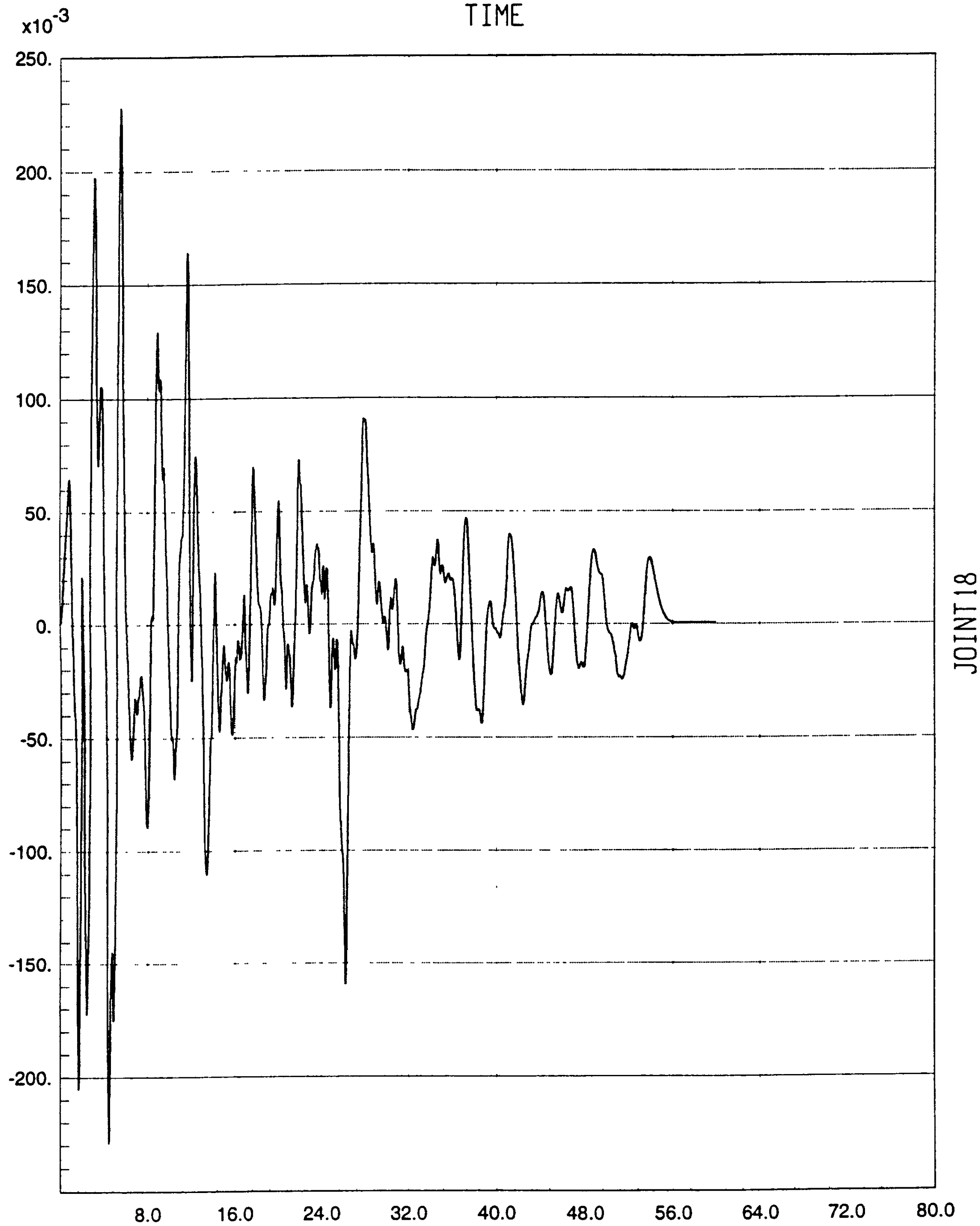
SAP2000

Time History Functions April 9,1998 16:27



SAP2000 v6.11 - File:frame3d1a - Kip-ft Units
Joint5: Joint 5 Displacement UX Vs Time
Min is -1.988e-01 at 4.5000e+00 Max is 1.918e-01 at 5.6000e+00

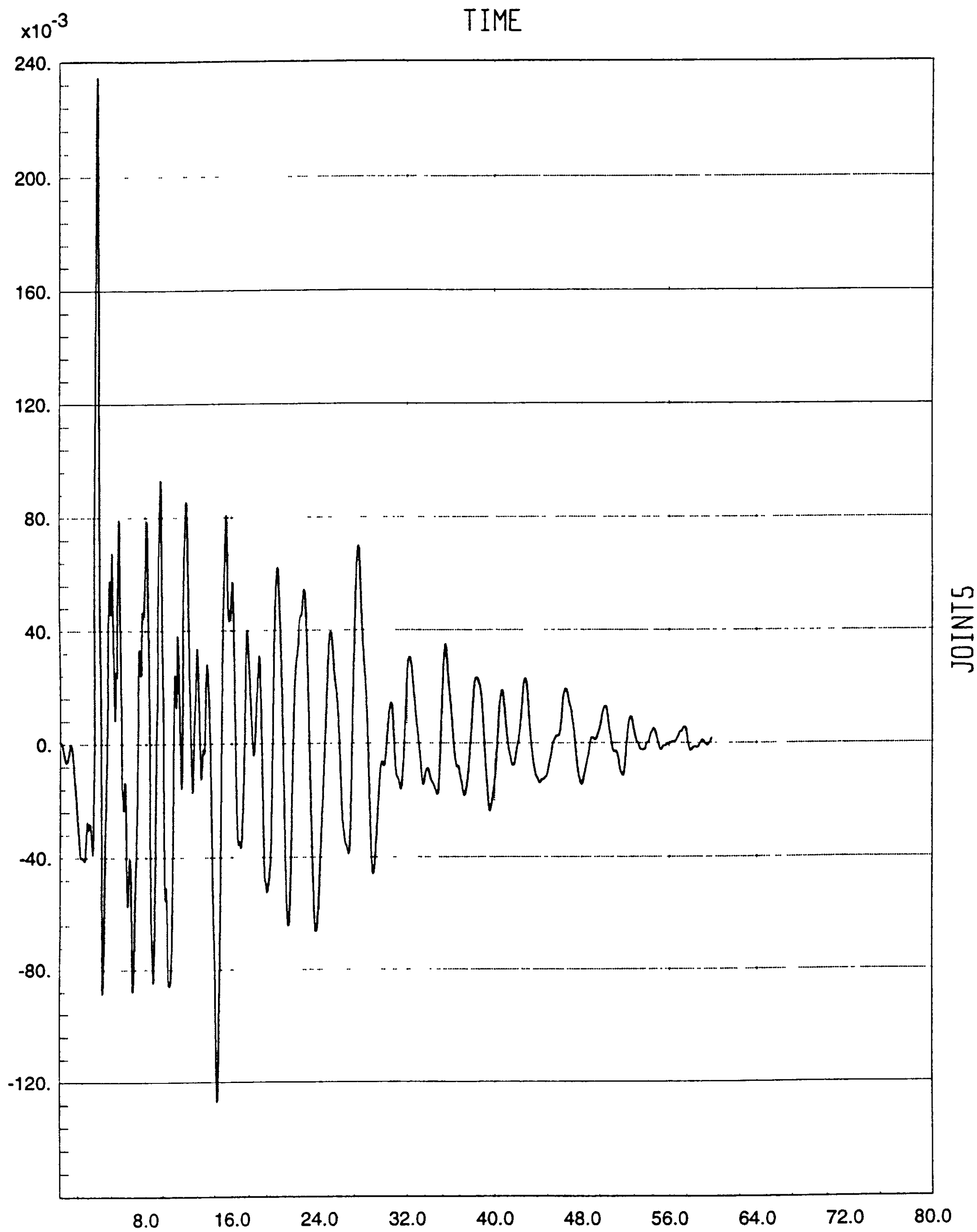
SAP2000 Time History Functions April 9,1998 16:27



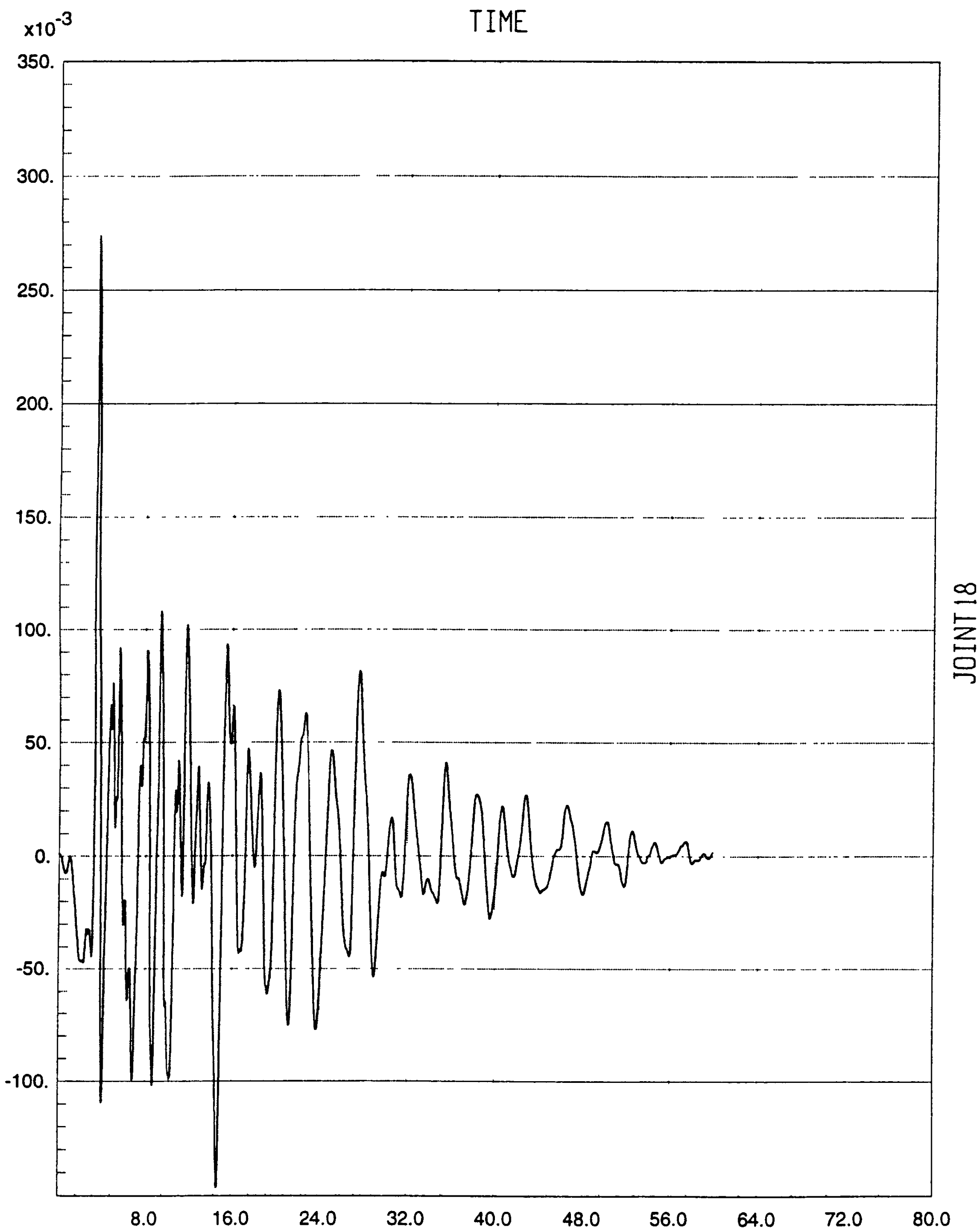
SAP2000 v6.11 - File:frame3d1a - Kip-ft Units
Joint18: Joint 18 Displacement UX Vs Time
Min is -2.300e-01 at 4.5200e+00 Max is 2.272e-01 at 5.6000e+00

SAP2000

Time History Functions April 9,1998 16:27

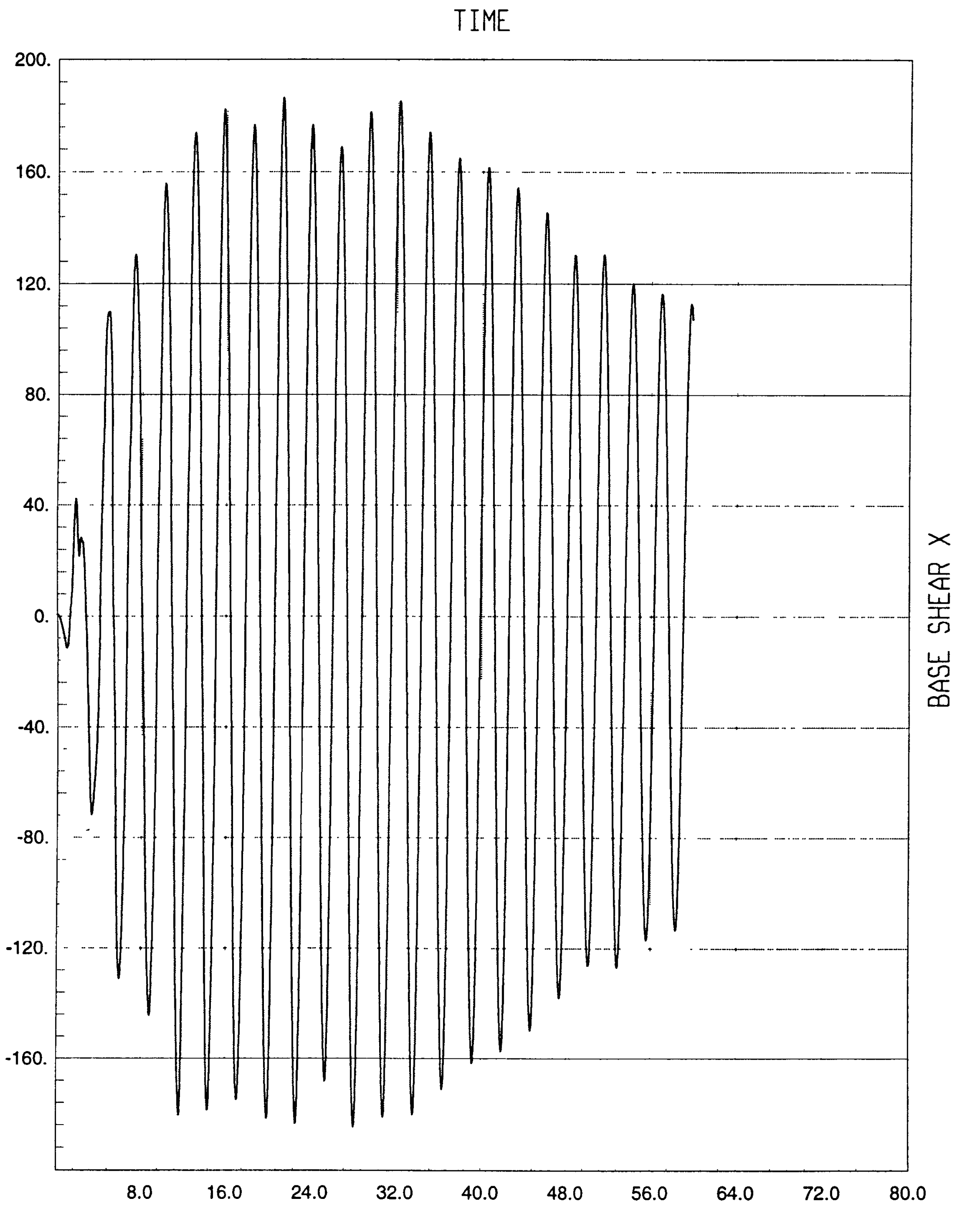


SAP2000 v6.11 - File:frame3d1a - Kip-ft Units
Joint5: Joint 5 Displacement UX Vs Time
Min is -1.279e-01 at 1.4700e+01 Max is 2.343e-01 at 3.5400e+00



SAP2000 v6.11 - File:frame3d1a - Kip-ft Units
Joint18: Joint 18 Displacement UX Vs Time
Min is -1.479e-01 at 1.4720e+01 Max is 2.735e-01 at 3.5600e+00

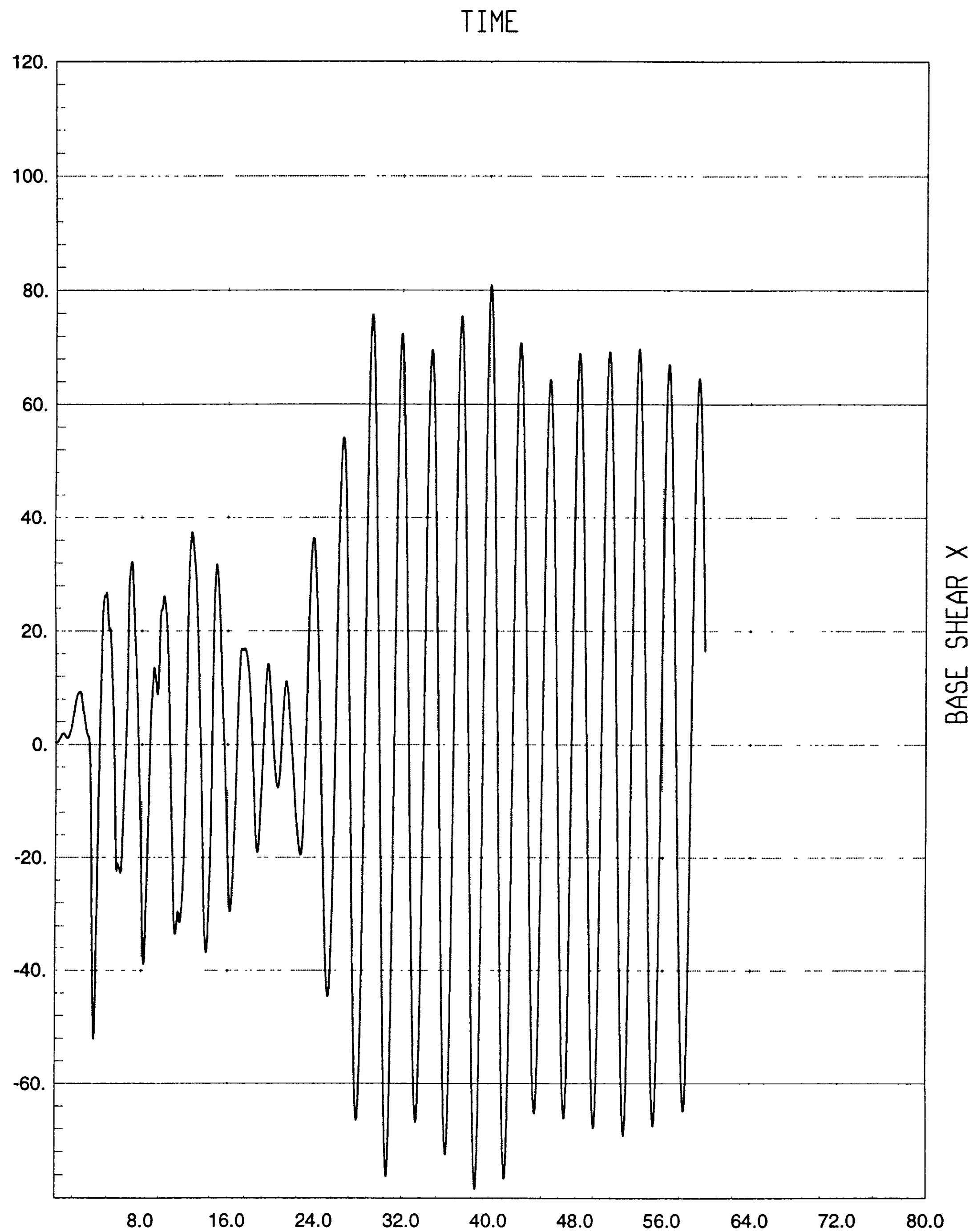
SAP2000 Time History Functions March 17,1998 23:44



SAP2000 v6.11 - File:frame3 - Kip-ft Units
Base Shear X: Base Shear X Vs Time
Min is -1.857e+02 at 2.8160e+01 Max is 1.861e+02 at 2.1340e+01

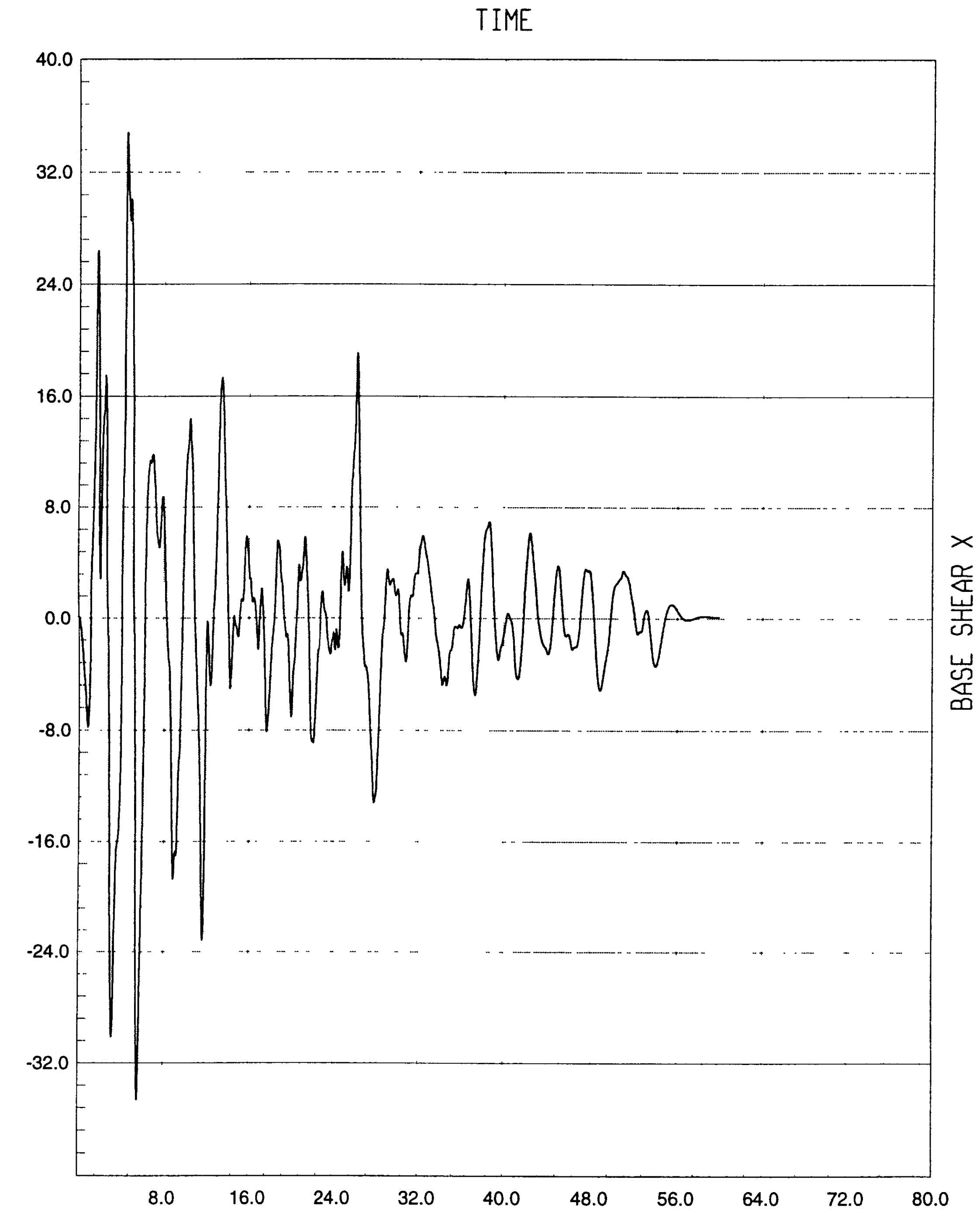
SAP2000

Time History Functions March 17,1998 23:58



SAP2000 v6.11 - File:frame3 - Kip-ft Units
Base Shear X: Base Shear X Vs Time
Min is -7.896e+01 at 3.8820e+01 Max is 8.074e+01 at 4.0160e+01

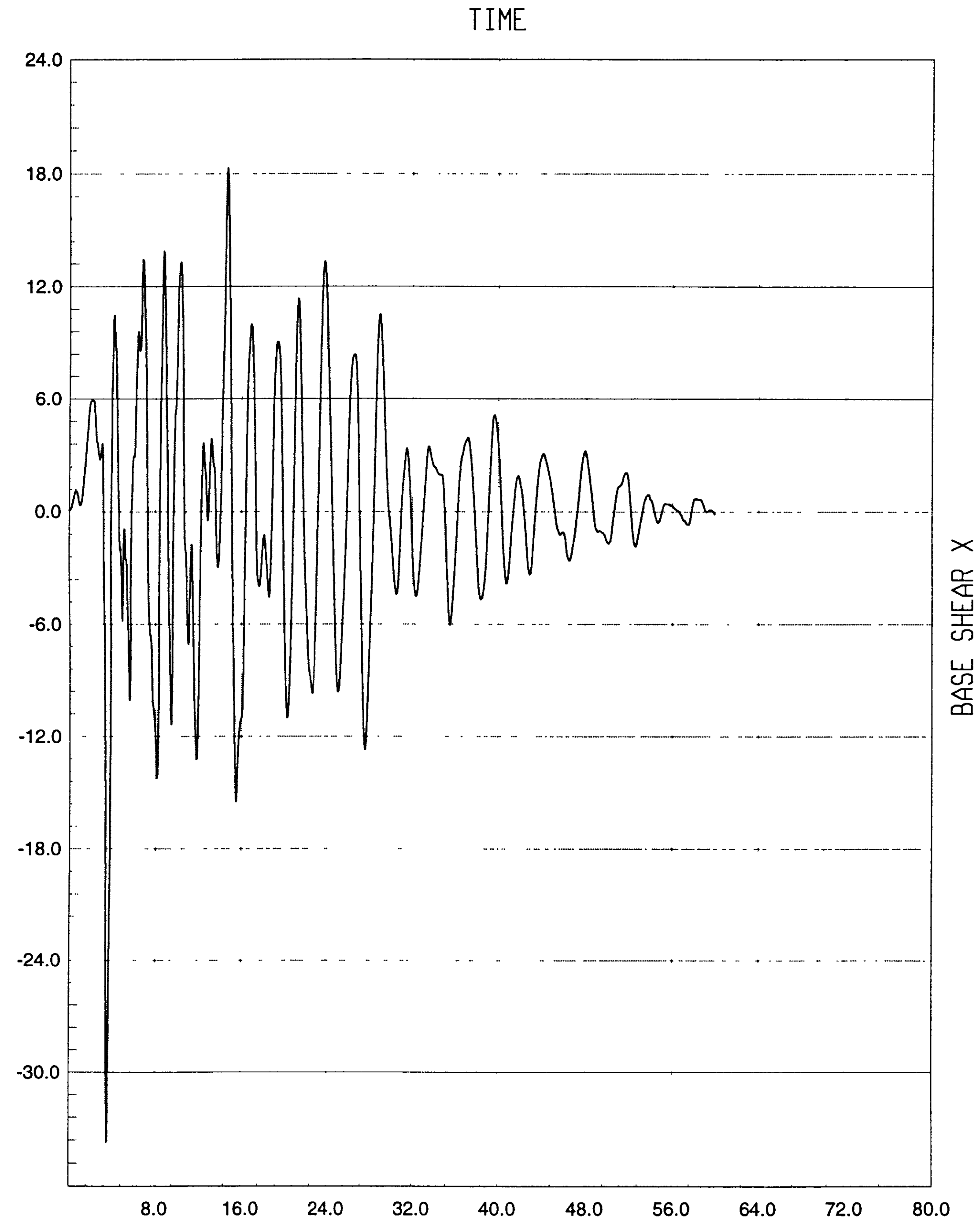
SAP2000 Time History Functions March 17,1998 20:26



SAP2000 v6.11 - File:frame3d2 - Kip-ft Units
Base Shear X: Base Shear X Vs Time
Min is -3.480e+01 at 5.6400e+00 Max is 3.474e+01 at 4.5400e+00

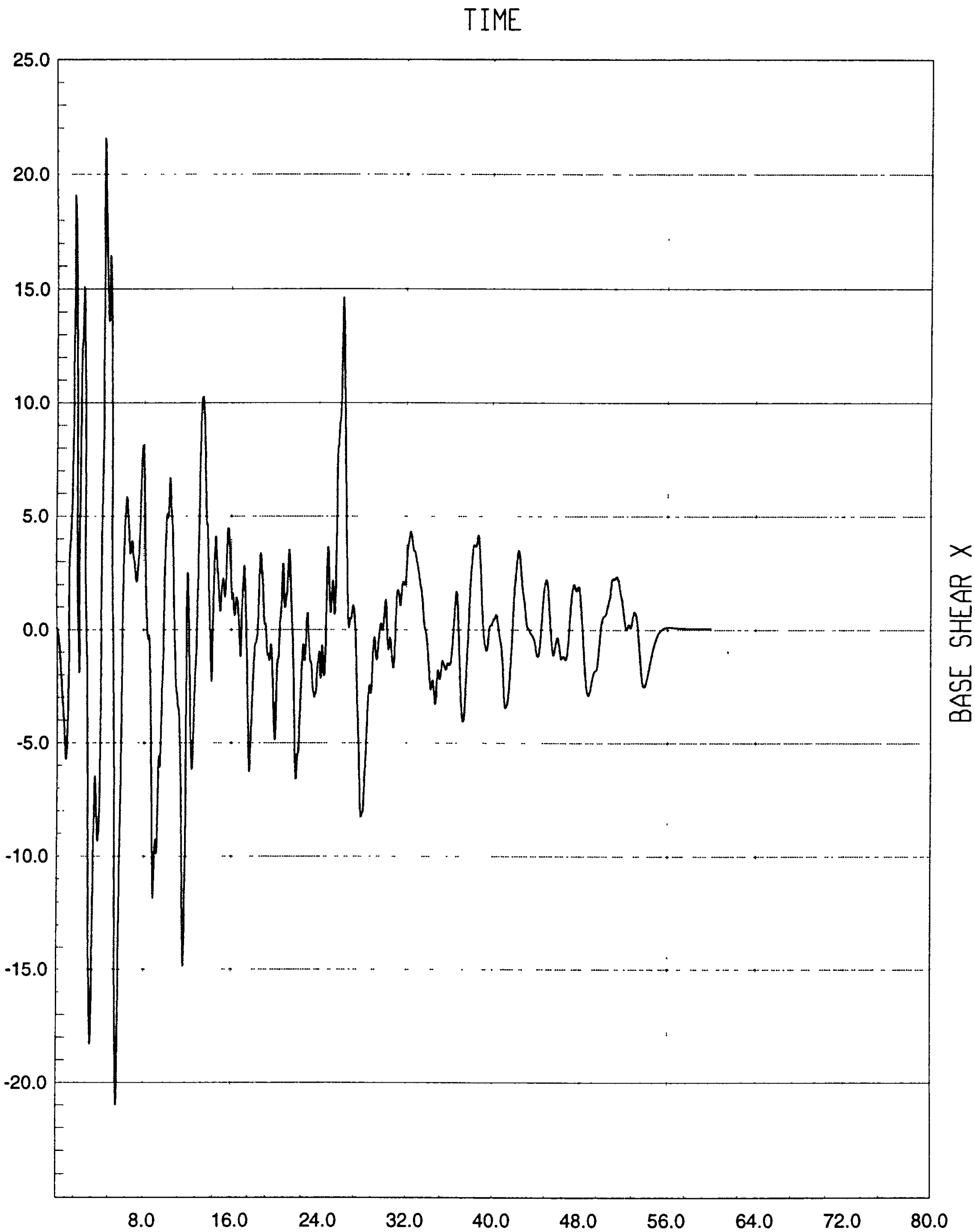
SAP2000

Time History Functions March 18,1998 0:59



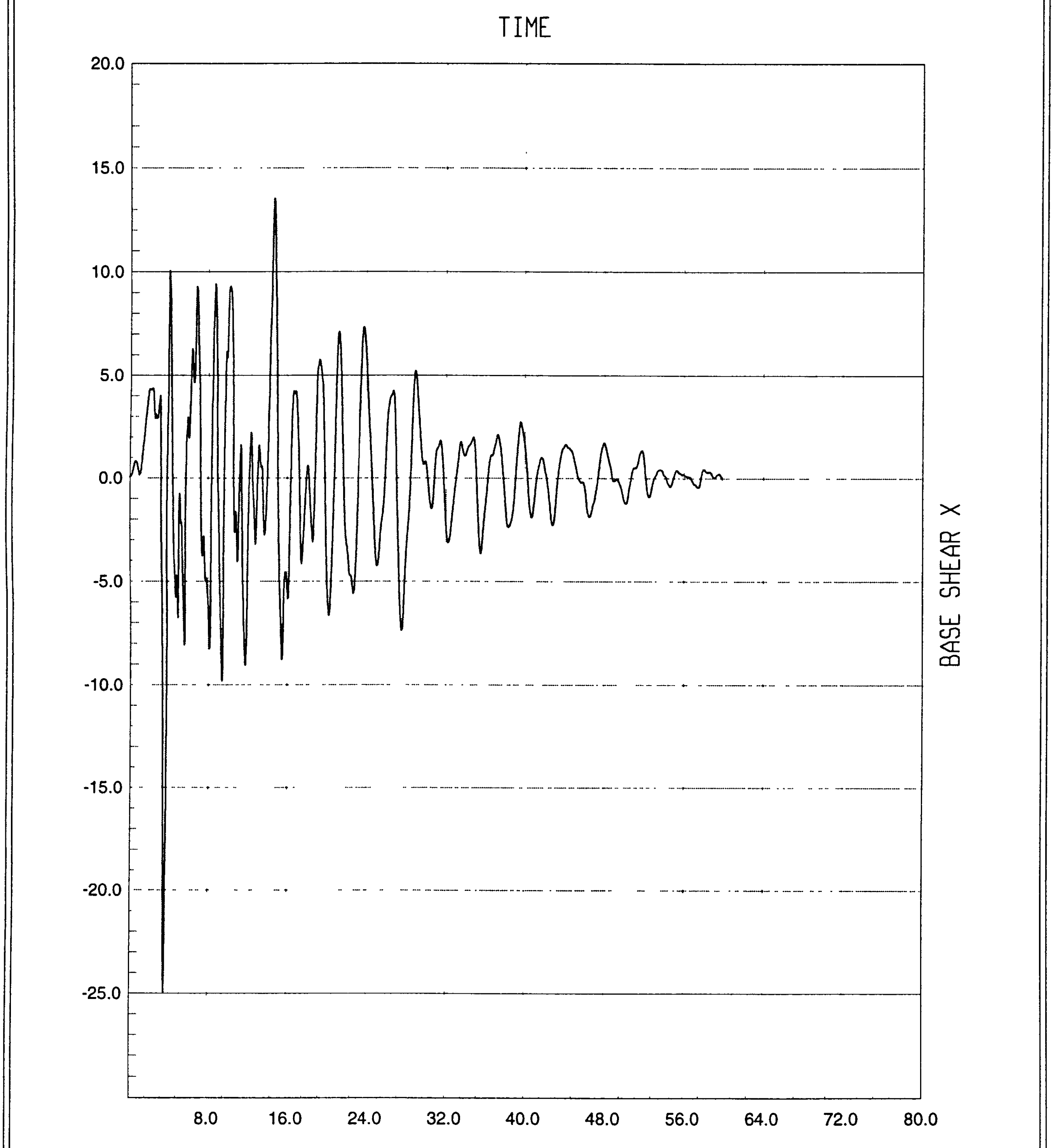
SAP2000 v6.11 - File:frame3d2 - Kip-ft Units
Base Shear X: Base Shear X Vs Time
Min is -3.387e+01 at 3.5800e+00 Max is 1.826e+01 at 1.4740e+01

SAP2000 Time History Functions March 18,1998 0:44



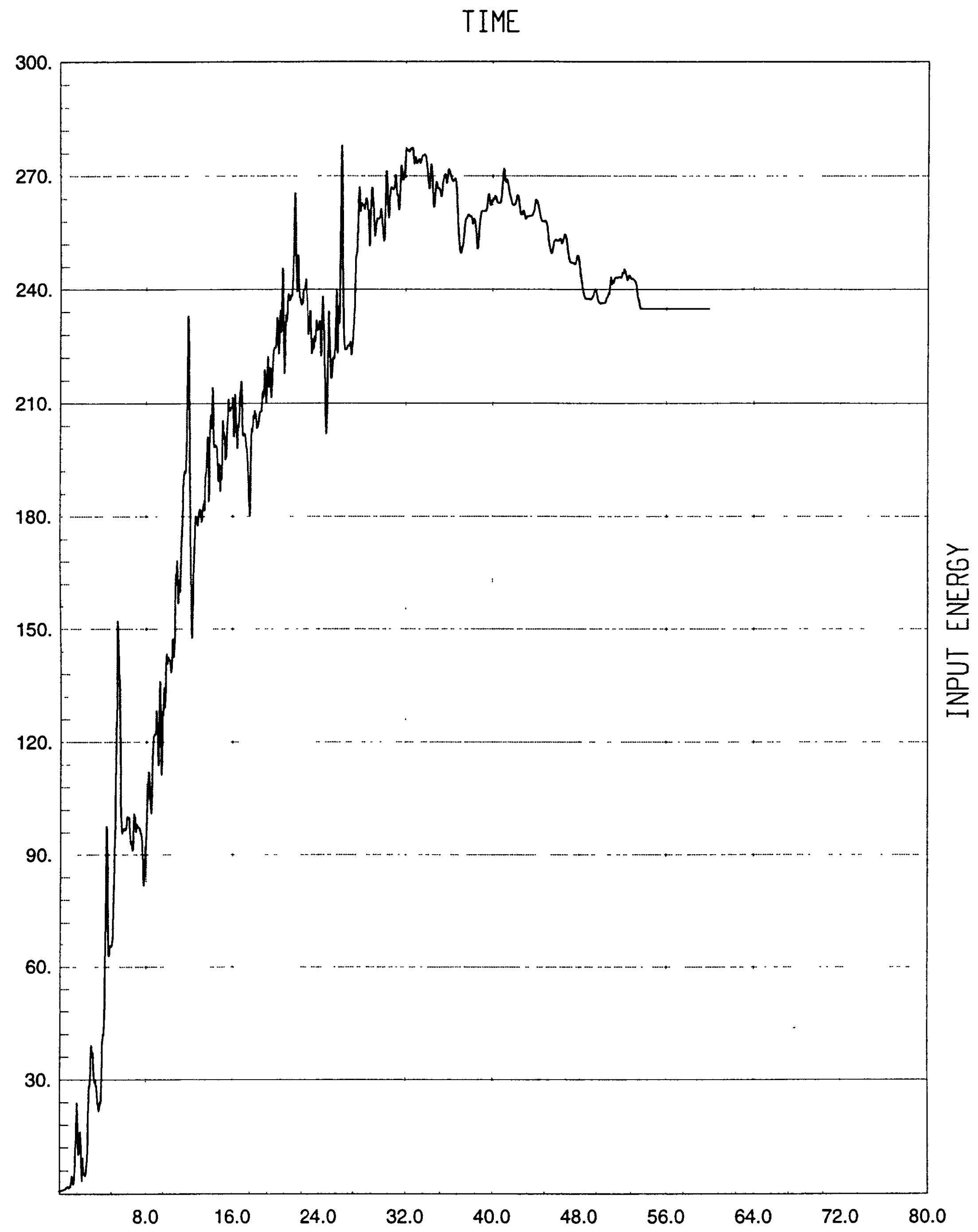
SAP2000 v6.11 - File:frame3d1 - Kip-ft Units
Base Shear X: Base Shear X Vs Time
Min is -2.111e+01 at 5.6000e+00 Max is 2.152e+01 at 4.5000e+00

SAP2000 Time History Functions March 18,1998 0:43

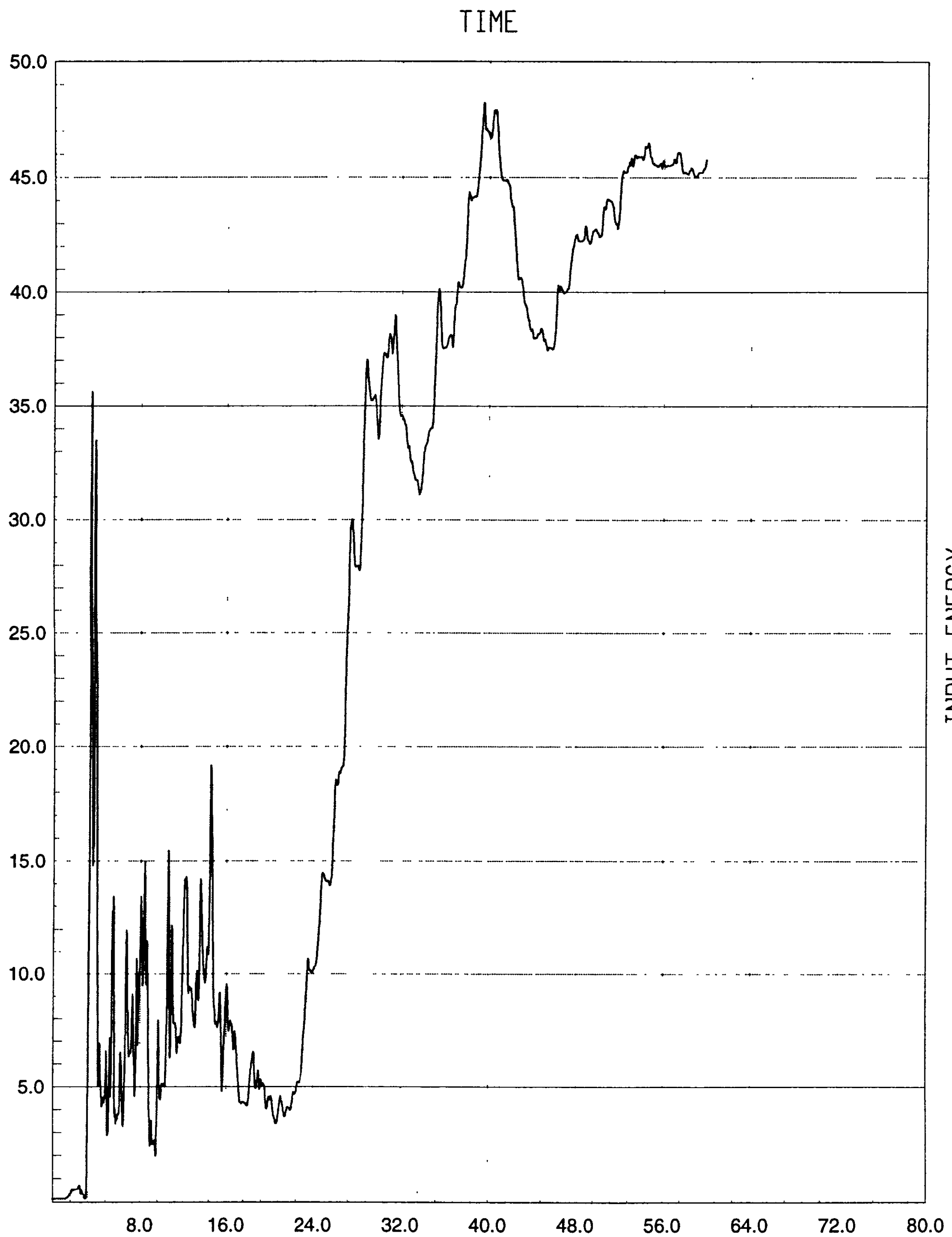


SAP2000 v6.11 - File:frame3d1 - Kip-ft Units
Base Shear X: Base Shear X Vs Time
Min is -2.507e+01 at 3.5600e+00 Max is 1.350e+01 at 1.4700e+01

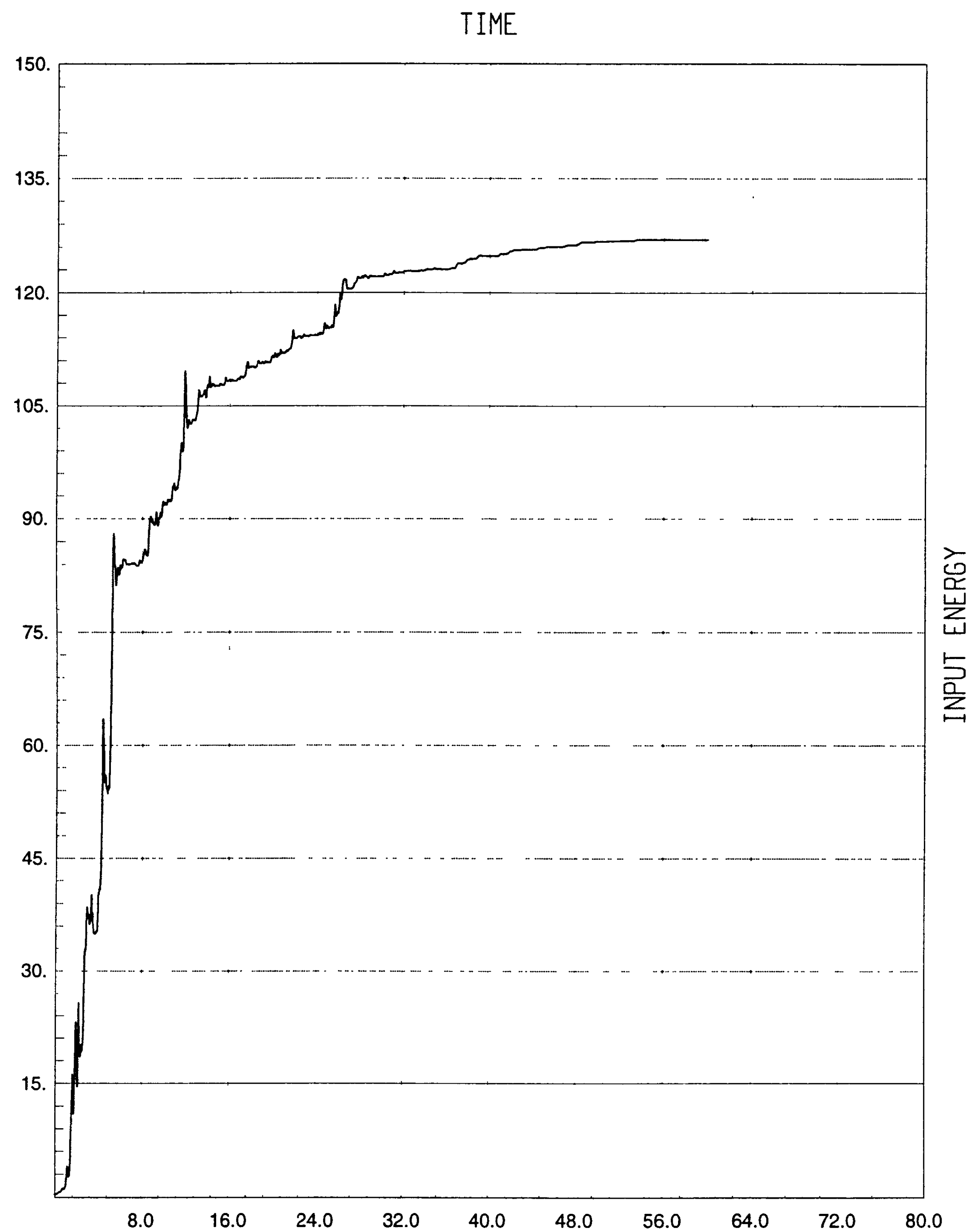
SAP2000 Time History Functions March 17,1998 20:04



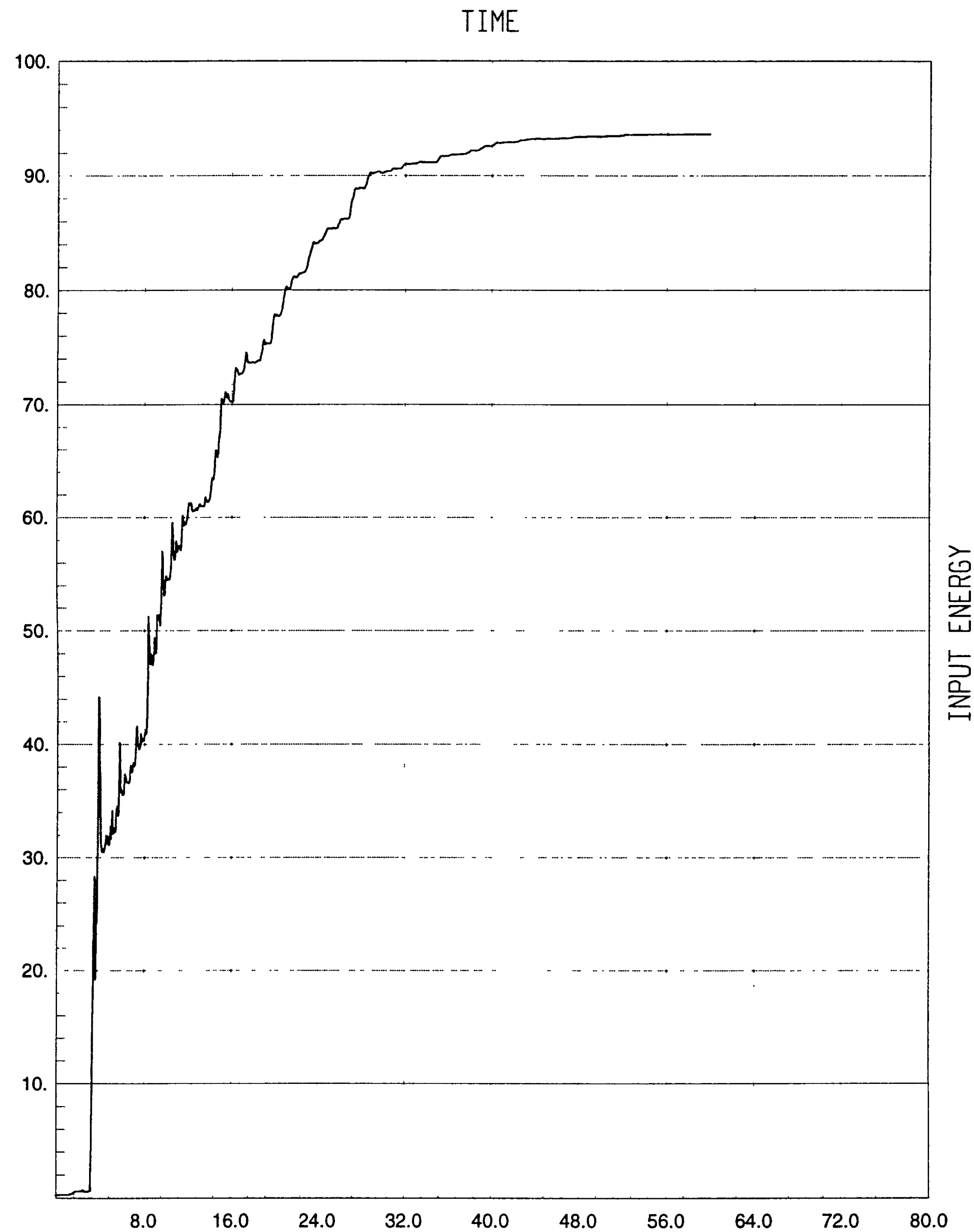
SAP2000 v6.11 - File:frame3 - Kip-ft Units
Input Energy: Input Energy Vs Time
Min is 0.000e+00 at 0.0000e+00 Max is 2.778e+02 at 2.6200e+01



SAP2000 v6.11 - File:frame3 - Kip-ft Units
Input Energy: Input Energy Vs Time
Min is 0.000e+00 at 0.0000e+00 Max is 4.819e+01 at 3.9560e+01

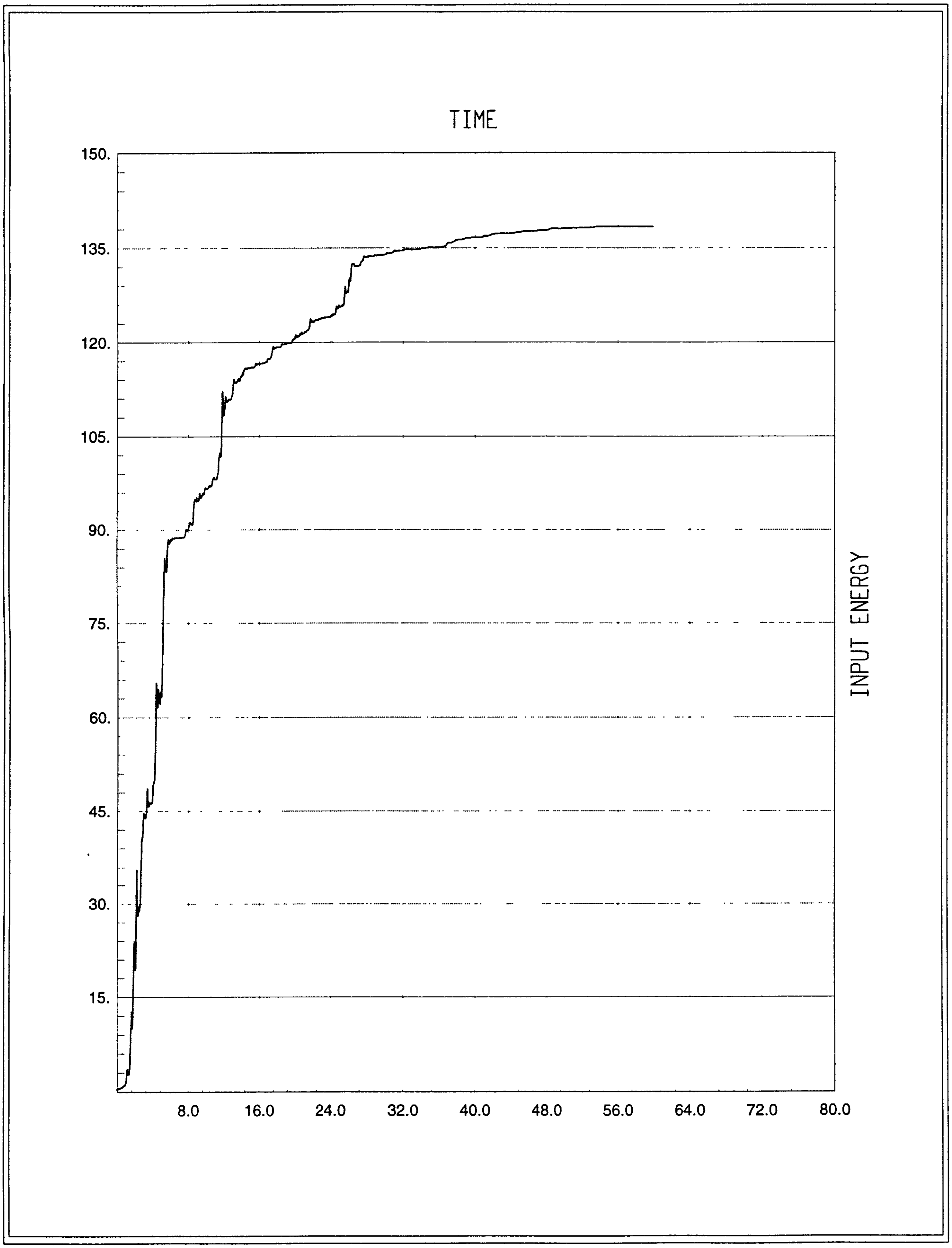


SAP2000 v6.11 - File:frame3d2 - Kip-ft Units
Input Energy: Input Energy Vs Time
Min is 0.000e+00 at 0.0000e+00 Max is 1.269e+02 at 5.3780e+01



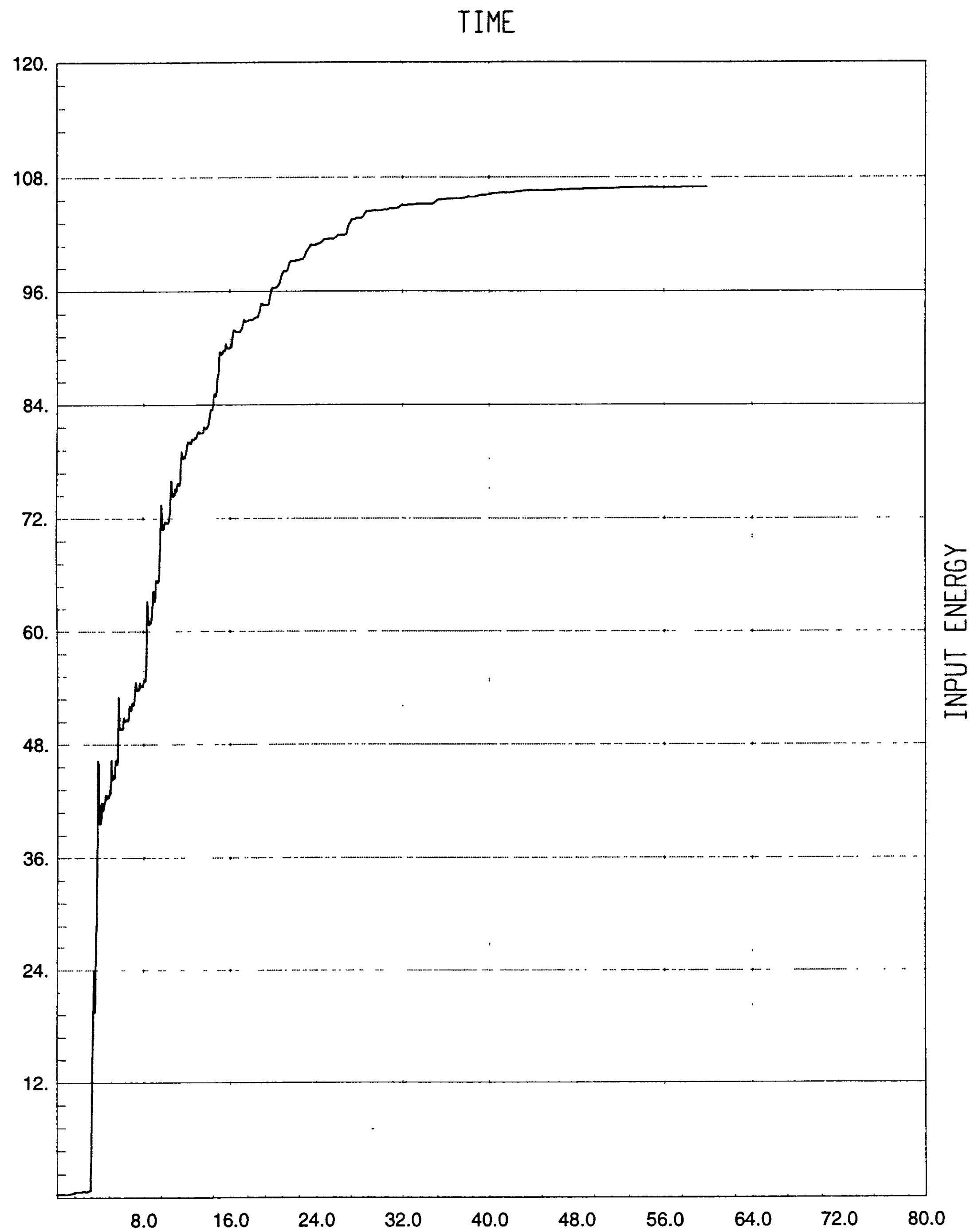
SAP2000 v6.11 - File:frame3d2 - Kip-ft Units
Input Energy: Input Energy Vs Time
Min is 0.000e+00 at 0.0000e+00 Max is 9.360e+01 at 5.7900e+01

SAP2000 Time History Functions March 17,1998 20:18



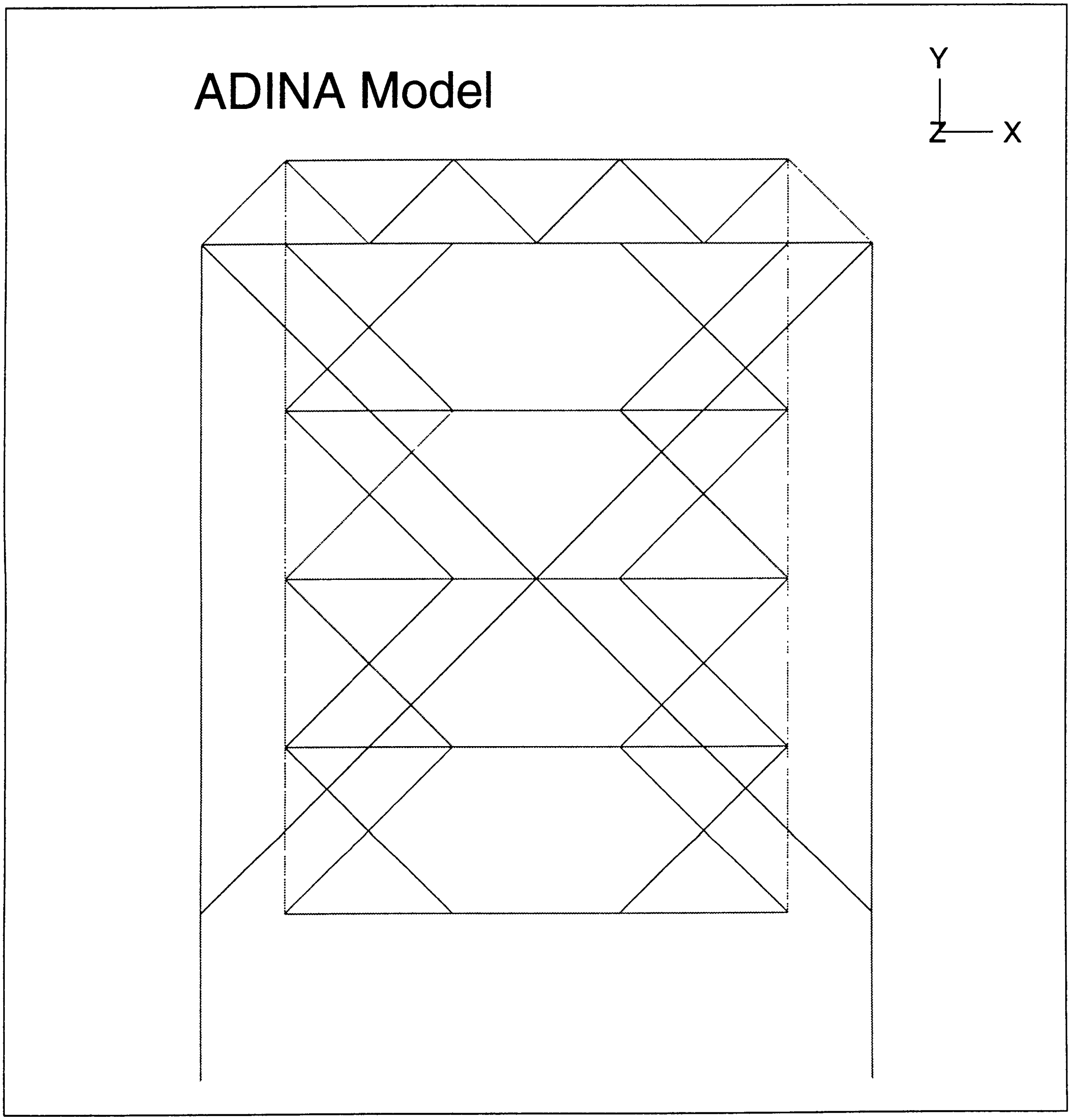
SAP2000 v6.11 - File:frame3d1 - Kip-ft Units
Input Energy: Input Energy Vs Time
Min is 0.000e+00 at 0.0000e+00 Max is 1.383e+02 at 5.3780e+01

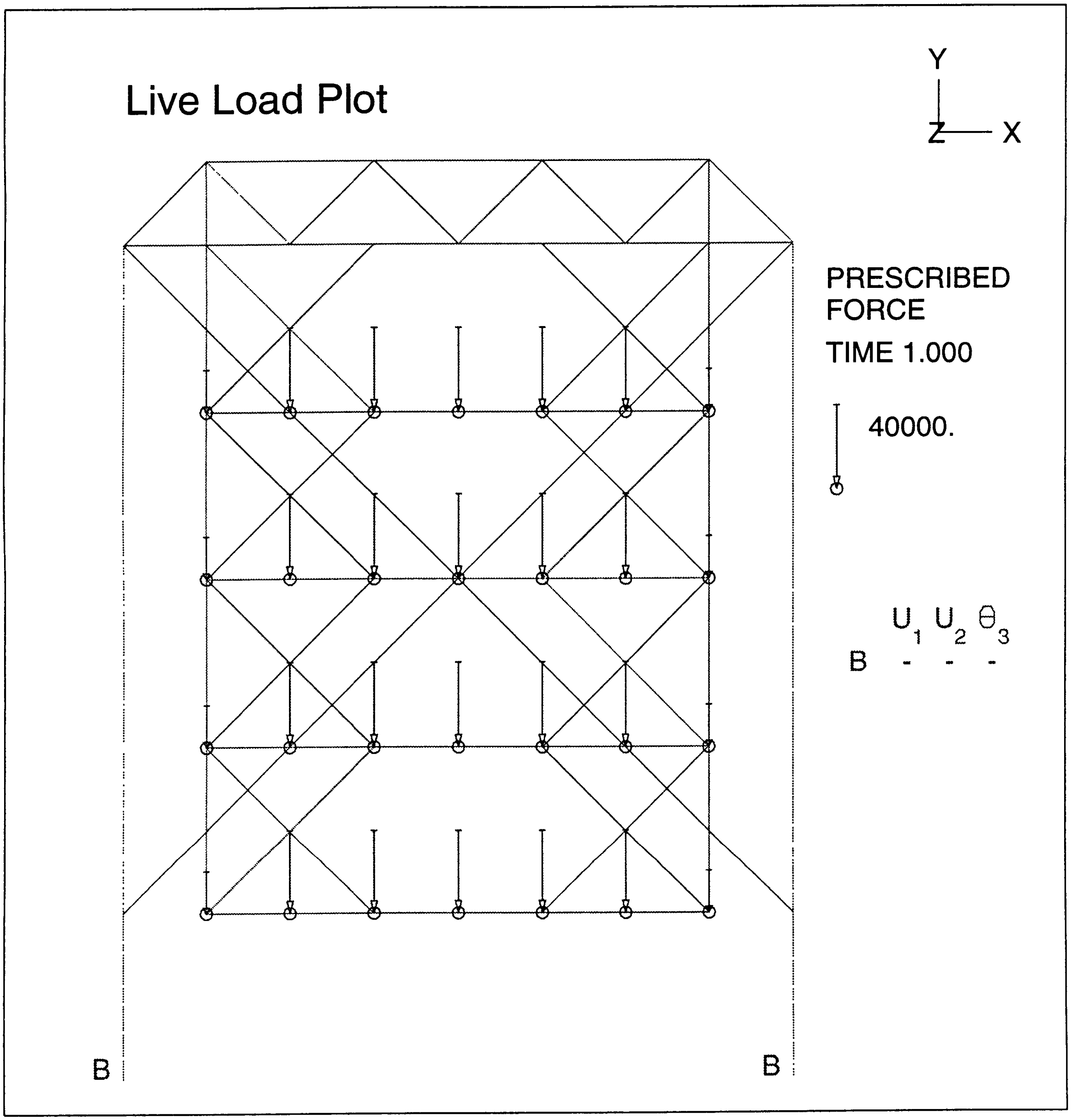
SAP2000 Time History Functions March 18,1998 0:40

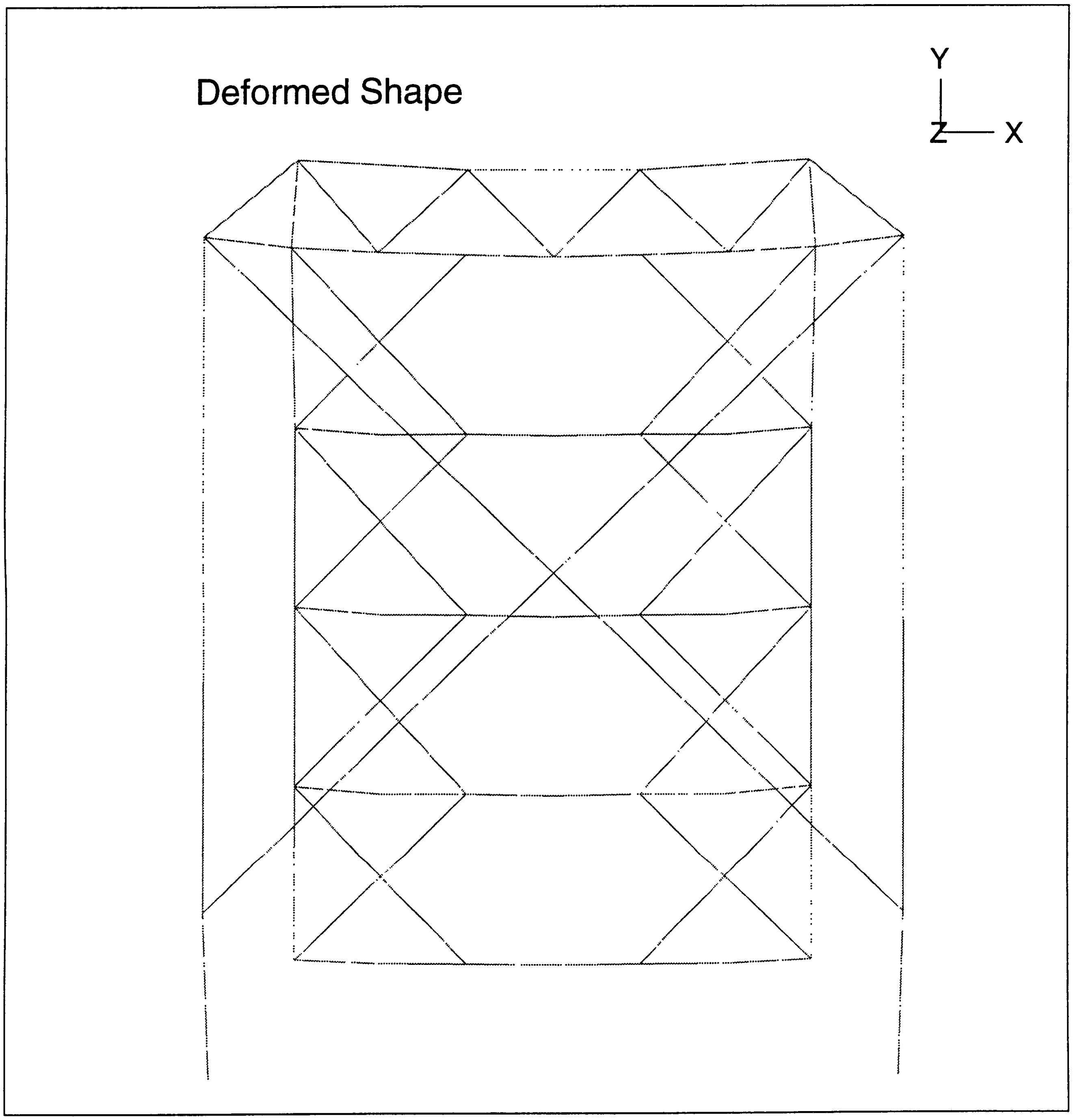


SAP2000 v6.11 - File:frame3d1 - Kip-ft Units
Input Energy: Input Energy Vs Time
Min is 0.000e+00 at 0.0000e+00 Max is 1.069e+02 at 6.0020e+01

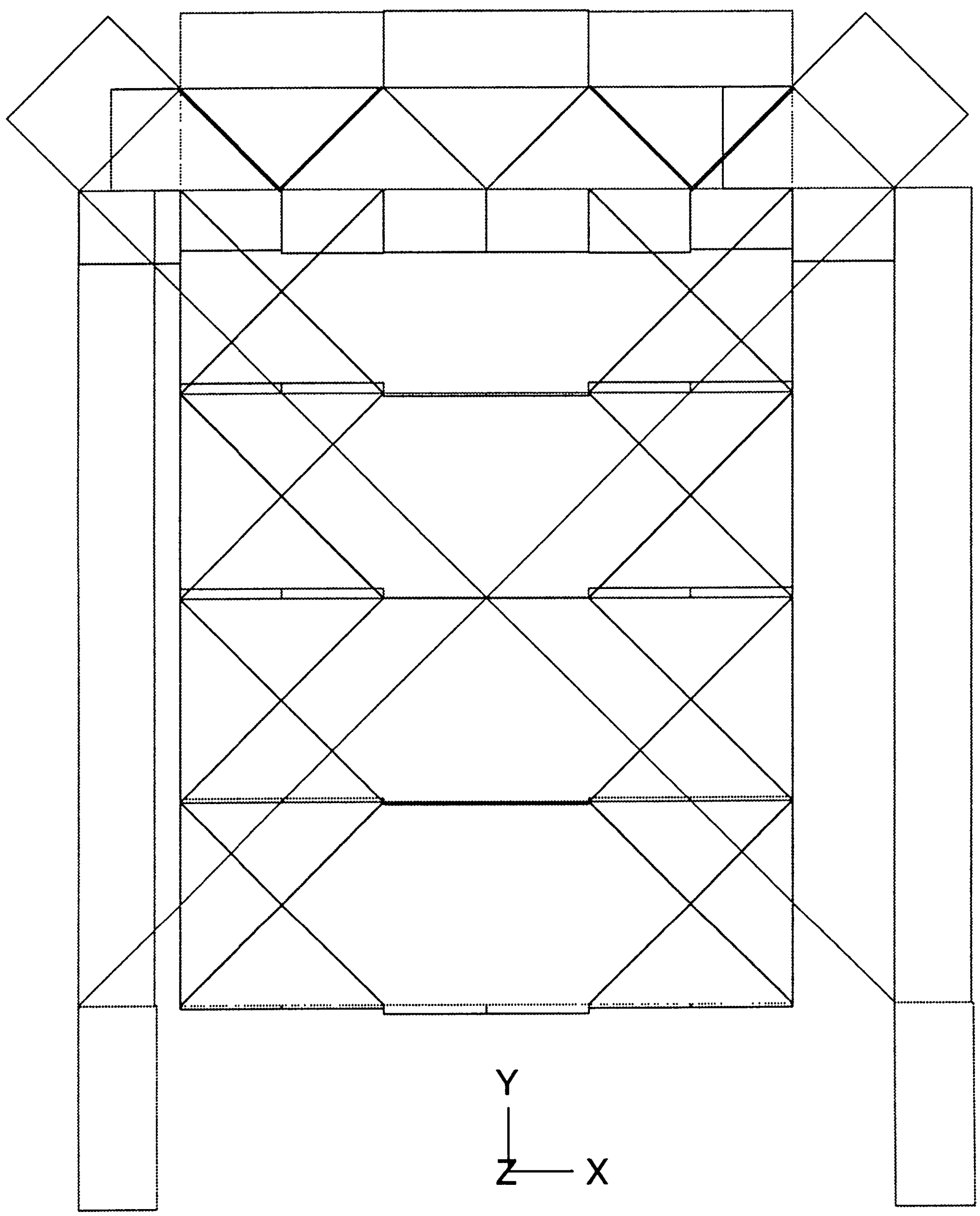
B. ADINA Output


ADINA Model
Y
Z
X

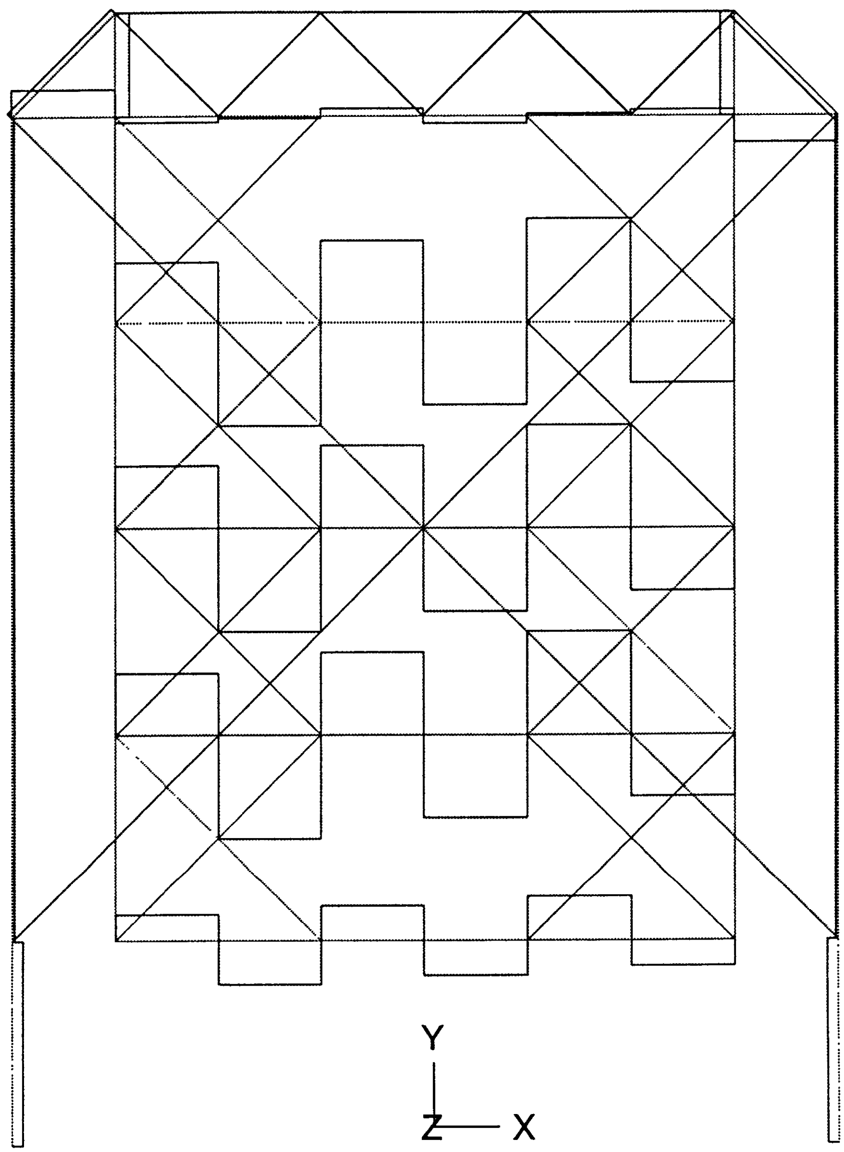

Live Load Plot
Y
Z
X
PRESCRIBED
FORCE
TIME 1.000
40000.
U_1 U_2 θ_3
B - - -
B
B


Deformed Shape
Y
Z
X

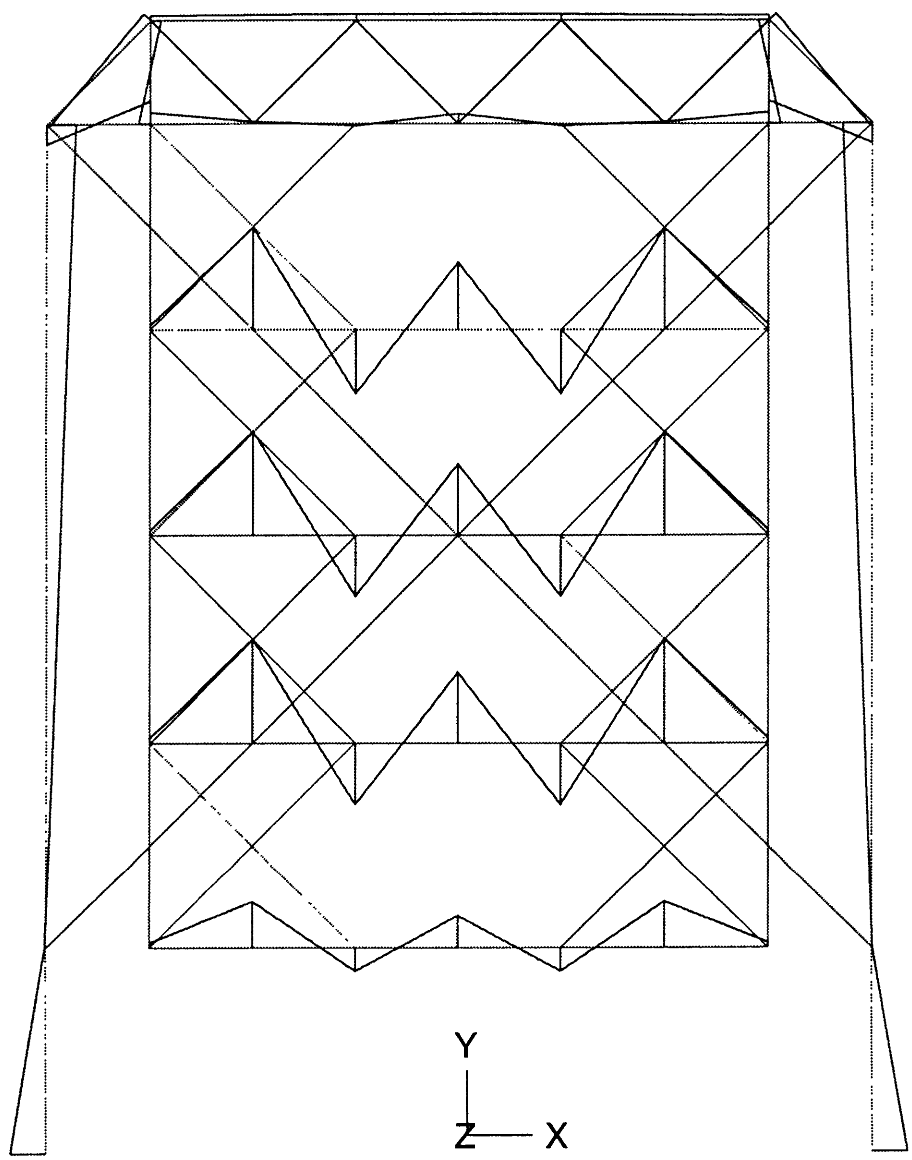
Axial Force Diagram

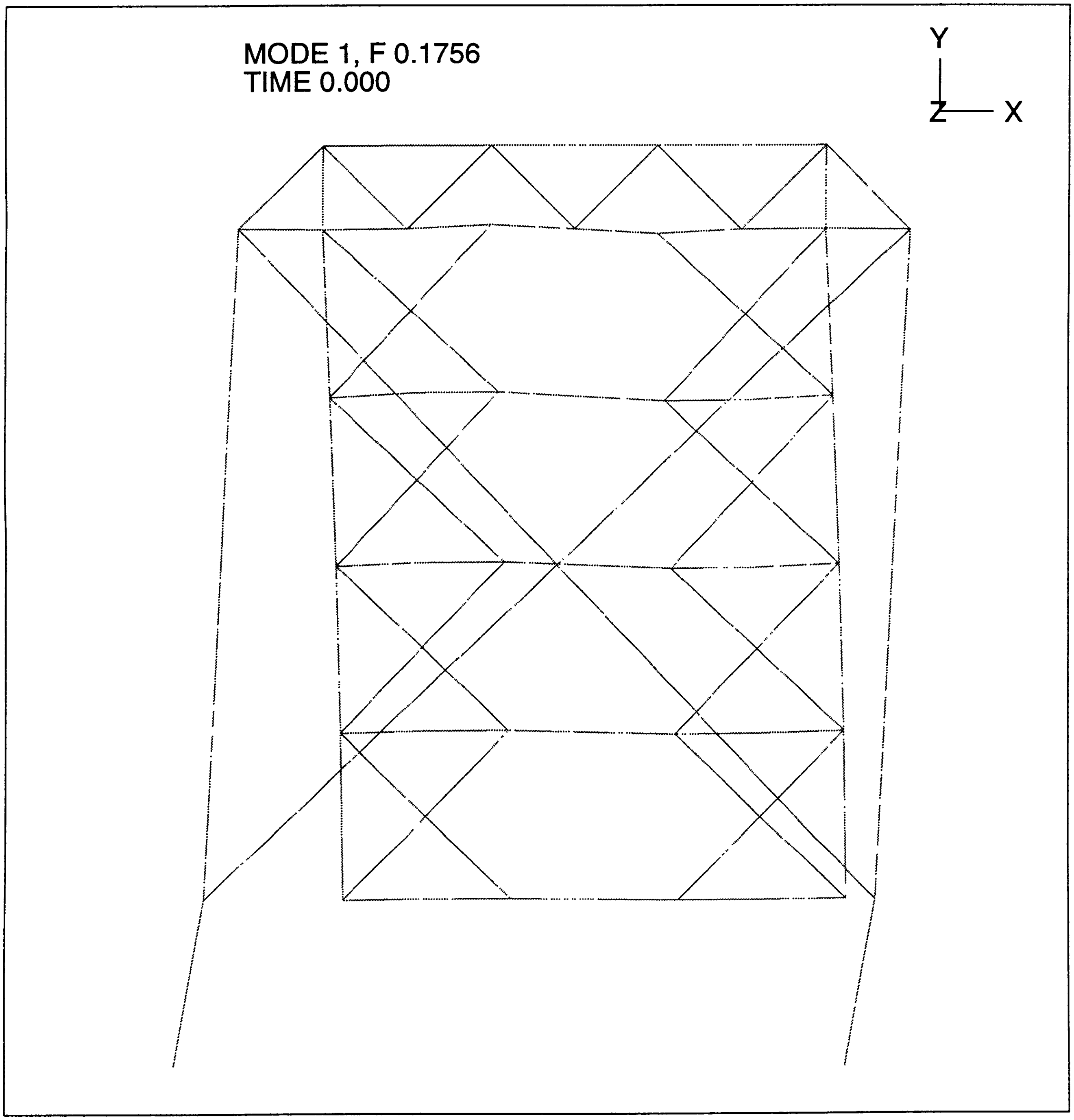


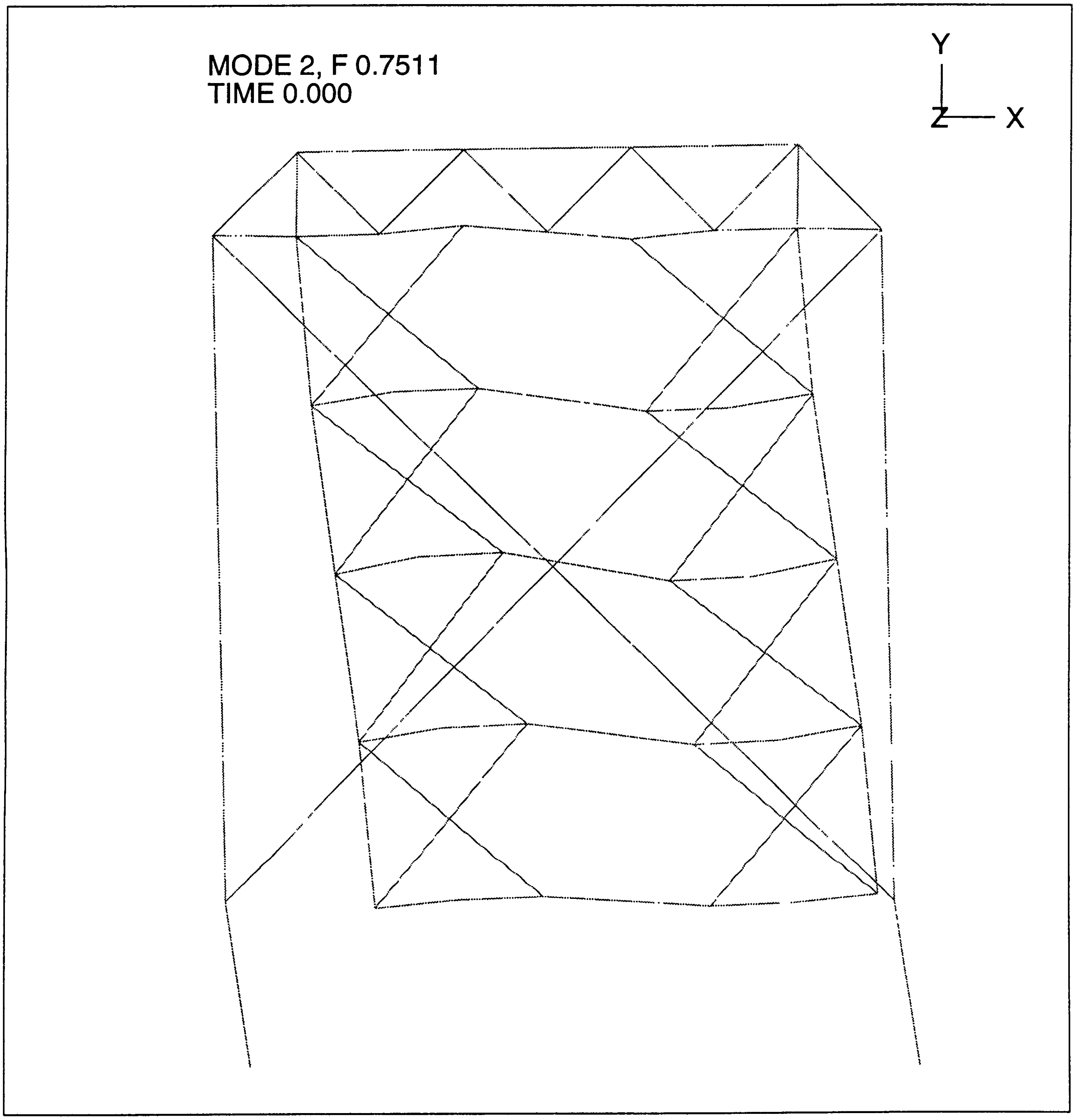
Shear Force Diagram

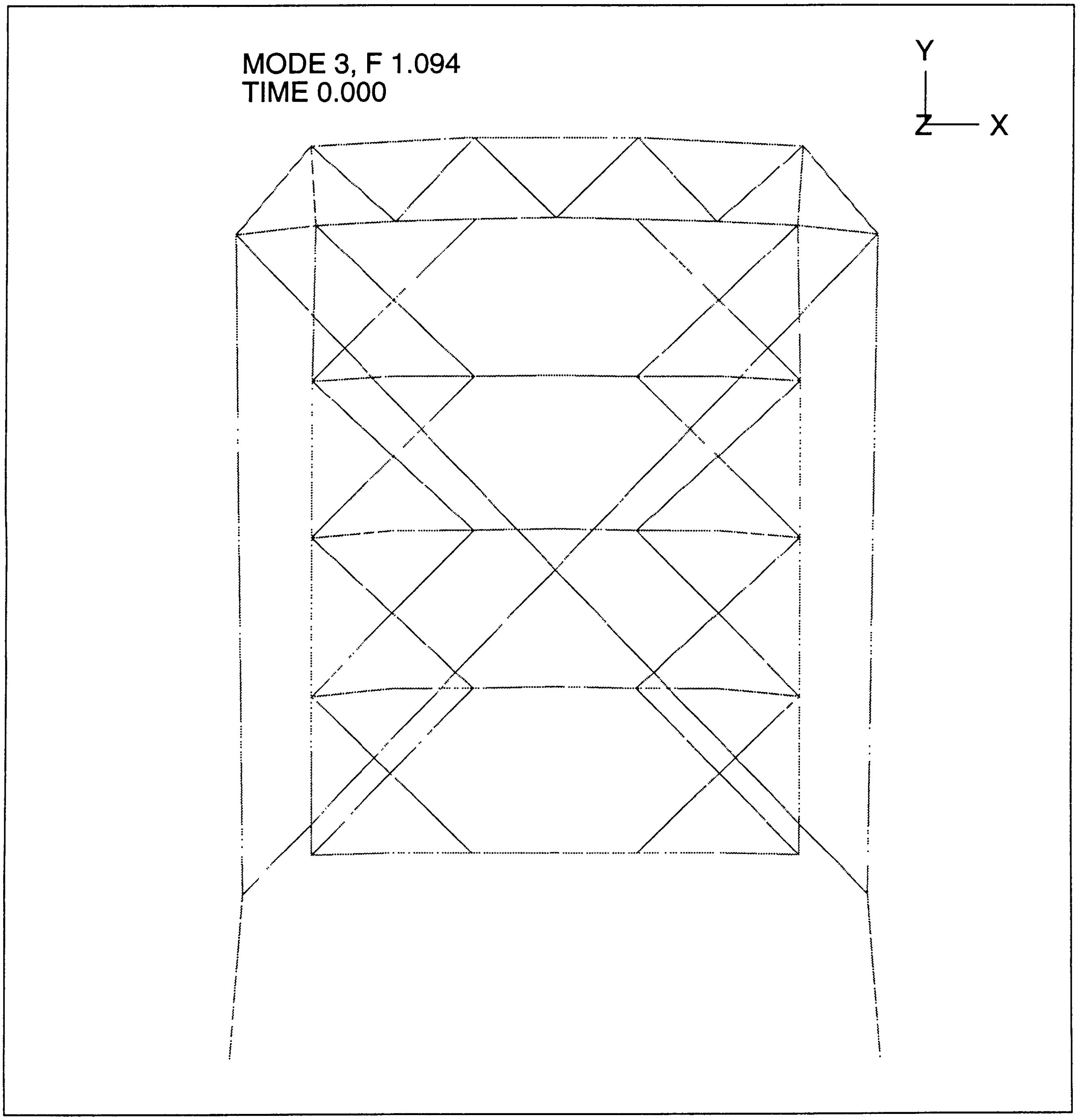


Bending Moment Diagram




MODE 1, F 0.1756
TIME 0.000
Y
Z
X


MODE 2, F 0.7511
TIME 0.000
Y
Z
X


MODE 3, F 1.094
TIME 0.000
Y
Z
X